GEOTECHNICAL CONSULTANTS, INC.
Geotechnical Engineering • Geology • Hydrogeology

Geotechnical Report
Muni Site Power Plant,
San Francisco, California

October 2005

Prepared for:

CH2M Hill

SF05019

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TABLE OF CONTENTS

Page

TABLES

FIGURES

PLATES

APPENDIX A – SUPPORTING GEOTECHNICAL DATA

INTRODUCTION

The City and County of San Francisco (CCSF), Public Utilities Commission (SFPUC) proposes to construct and operate a simple-cycle power plant, referred to throughout this report as the Muni Site Power Plant, on the eastern four-acre portion of the Muni Metro East Light Rail Vehicle Maintenance and Operation Facility. The power plant is proposed as part of SFPUC's Electrical Reliability Project (ERP), and will improve the CCSF's electricity reliability and replace aging and polluting in-city generation facilities (CH2M Hill, 2005). We understand that this report will be included as part of the bid package for a design-build contract and will also be submitted for California Energy Commission review.

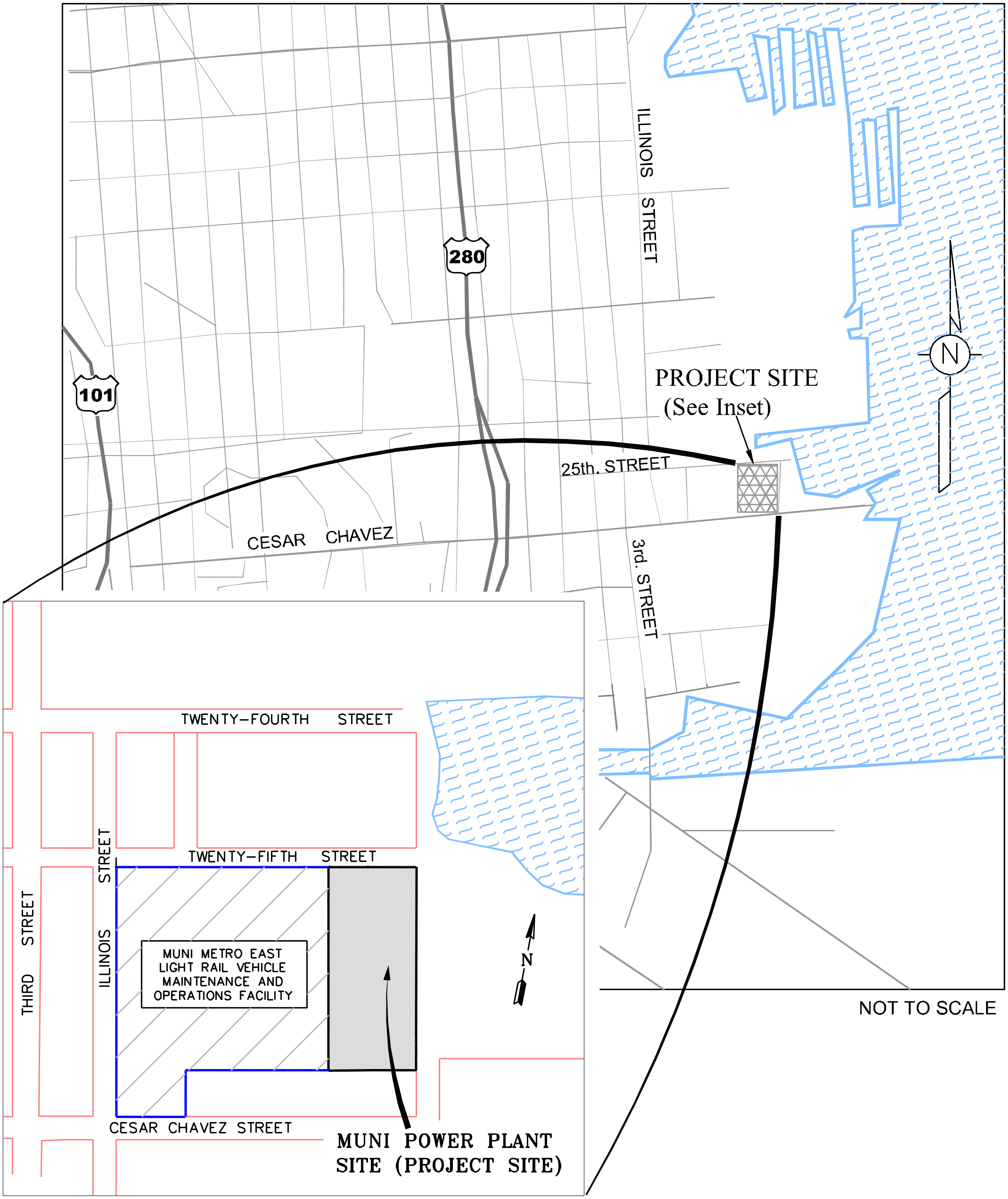
This report presents the findings and design recommendations resulting from our design-level geotechnical investigation performed at the request of PB Power and CH2M Hill for the proposed Muni Site Power Plant. The project site is bounded by Cesar Chavez Street to the south, 25th Street to the north, and is approximately 750 feet east of Illinois Street, in San Francisco, California. The location of the project site is shown on Figure 1 - Location Map. The site is currently owned by the Port of San Francisco and is being leased by the Municipal Railway of San Francisco (Muni). Hetch Hetchy Water and Power, SFPUC, plans to construct a power plant at the site as part of their ERP. This geotechnical report was prepared for PB Power and CH2M Hill as part of this effort, and was carried out based on a proposed schematic layout and loads of power plant structures provided by PB Power.

This report provides an overview of existing geotechnical/geologic conditions at the proposed power plant site and geotechnical design parameters for the proposed facilities. The geotechnical site conditions presented herein are based on our field exploration as well as literature review from available geotechnical/geologic reports in the project vicinity. This report does not include environmental site characterization, hazardous materials testing, or other environmental services.

PROPOSED DEVELOPMENT

The proposed power plant project entails the construction of three LM6000 combustion turbine generators, SCR/CO catalyst systems, and stacks; four fuel gas compressors and fuel gas cooling radiators; two water storage tanks; a switchyard; a chiller and cooling tower; a two story plant operations building; and numerous appurtenant structures. Access to the site will be at the northwestern corner of the site, at the eastern end of 25th Street. A plan of the proposed layout of structures is shown on Plate 1 – Site Plan, Muni Site, SFPUC ERP Power Plant. Design loads associated with the main power plant structures were provided to us by PB Power, and are as follows:

FIGURE 1
LOCATION MAP



SF05019-2

•	Treated Water Storage Tank	5,480 kips
•	DI Water Storage Tank	2,740 kips
•	Combustion Turbine Generator	750 kips
•	SCR/CO Catalyst System and Stack	400 kips
•	Generator Step-Up (GSU) Transformer	220 kips
•	Fuel Gas Compressor	45 kips each

PREVIOUS REPORTS

To assist us in our analyses, we reviewed geotechnical information from previous reports in the project vicinity to evaluate past findings and fill any data gaps to assist us in developing our field exploration. In addition to published geologic, geotechnical, and seismic references and maps, the following consultant's reports were reviewed:

- "Geotechnical Report, Potrero Power Plant, San Francisco, California", Geotechnical Consultants, Inc., June, 2004.
- "Final Geotechnical Study Report, MUNI Metro East Light Rail Vehicle Maintenance and Operations Facility", AGS, Inc., August 1999.

WORK PERFORMED

The scope of work for this project was developed based on (1) correspondence and discussions with Steve Brock of PB Power and John Carrier of CH2M Hill; (2) drawings of the power plant preliminary design layout and the power plant site in relation to the Muni Metro East facility, as provided by PB Power; and (3) a review of available geologic and geotechnical information.

We performed the following work for this geotechnical evaluation:

1. **Exploratory Drilling.** Explored subsurface conditions by means of fifteen rotary wash borings, B-1 through B-15. All boring logs are appended to this report. The boring locations are shown on Plate 2 – Field Exploration Plan. The following table shows the drilling dates, ground elevations, and depths of the borings.

TABLE 1 – BORING DATES, ELEVATIONS, AND DEPTHS

Boring	Date Drilled	Ground Elevation (feet) [1]	Depth (feet)
B-1	7/23/05	12.0	100.5
B-2	7/30/05	13.5	101.5
B-3	7/23/05	13.5	32.5
B-4	7/23/05	10.5	168.2
B-5	7/27/05	11.0	100.5
B-6	7/27/05	11.0	100.5
B-7	8/2/05	11.5	101.0
B-8	7/28 and 7/29/05	11.5	101.5
B-9	7/29 and 8/1/05	12.5	100.9
B-10	7/25/05	12.5	31.5
B-11	7/26/05	13.5	101.5
B-12	7/26/05	14.0	101.5
B-13	7/25/05	14.5	33.0
B-14	7/22 and 7/25/05	12.5	101.5
B-15	7/20 and 7/21/05	14.5	150.0

[1] Elevations are based on interpolation between survey points and/or elevation contours from survey map provided by PB Power, and are estimated to the nearest 0.5 feet. Elevation datum is NAVD 1988.

Soils samples were recovered by split-spoon SPT, 2-½ inch diameter sleeve samples using a split-barrel sampler, and Shelby Tube. Samples were visually classified and submitted for testing in the laboratory. Boring logs and laboratory test data are presented in Appendix A - Supporting Geotechnical Data.

2. **Laboratory Testing.** Performed laboratory tests, including moisture, density, grain size distributions, Atterberg limits, unconfined compression strength, triaxial undrained shear, consolidation, R-value, and corrosion on selected soil samples to measure pertinent index and engineering properties. Details of the laboratory testing program test results are presented in Appendix A.

3. **Engineering Analysis.** Analyzed findings to develop geotechnical recommendations for seismic design criteria, earthwork, foundations, lateral earth pressures, pavement, and corrosion.

4. **Vibration Study.** Analyze and evaluated vibration levels emitted from gas compressors at the power plant, and transmitted to the Muni Metro East facility. Evaluated foundation design for the gas compressors to limit soil excitation at the source of the vibrations.

5. **Site Specific Seismic Study.** Conducted a site specific seismic hazards analysis for

horizontal ground motions with a probability of exceedance of 10 percent in 50 years and 2 percent in 50 years. Constructed a design response spectra in accordance with FEMA 356.

6. **Geophysics Testing.** Conducted geophysical testing at the site to evaluate the presence and distribution of subsurface obstructions in the upper 10 feet of fill and to determine the dynamic properties of the soils for our vibration study.

7. **Report.** Prepared this report presenting our geotechnical findings, conclusions, and recommendations for the design and construction of the proposed project.

FINDINGS

SITE CONDITIONS

The site of the proposed power plant encompasses approximately four acres approximately 750 feet east of Illinois Street, and is bounded by 25th Street to the north and Cesar Chavez Street to the south. The project site is situated in an area reclaimed from the San Francisco Bay, and is approximately 500 feet from the Bay shoreline. The property is owned by the Port of San Francisco and is leased by Muni. The property directly to the south of the project site is occupied by a truck rental facility. The area to the east is being utilized as a holding yard for truck trailers. The Muni Metro East Light Rail Vehicle Maintenance and Operations Facility is proposed to be constructed on the vacant 13 acres to the west of the project site.

Currently, Pacific Cement occupies the northern portion of the project site, and operates a concrete batch plant, which extends from 25th Street to a chain-linked fence approximately 250 feet to the south. The entrance to the batch plant is at the eastern end of 25th Street. Various appurtenant structures, such as two hoppers, several trailers, and conveyor belts are situated in this area of the project site. Construction trailers are located along the western property line. The hoppers and the main batch plant facilities are located in the center of the area. Concrete walls and k-rails form dividers for stockpiles in the eastern portion of the plant. The presence of truck and heavy equipment traffic impacted our boring locations, drill rig access, and drilling schedule. A dirt and gravel access road circles the plant. The ground surface in the majority of the plant area is hardened concrete residue over soil or gravel. Concrete slabs have been constructed in some areas of the site for equipment and truck access, and for plant facility and trailer foundations. Standing water about one foot deep from plant processes was present over approximately one fourth of the batch plant area.

The middle portion of the project site is undeveloped, and extends from the concrete batch plant to a chain-linked fence approximately 180 feet from the southern boundary of the project site. The ground surface is mostly gravel and soil with some vegetation. Square concrete piles were stockpiled in the northeastern portion of this area and are presumed to be for the construction of the neighboring Muni Metro East Facility. Vehicle access to this area is through a gate on the eastern property line separating the project site from the truck trailer yard.

The southern portion of the site extends from the southern property line north approximately 180 feet to a chain-linked fence. A four-wide construction trailer occupies the western portion of this area of the project site. Two concrete slabs at grade, each approximately 10 feet square, are located in the southeastern corner of the area.

The site was surveyed, and a topographic map was provided to us by PB Power. All elevations referred to in this report are with respect to NAVG 1988 datum. The project site and surrounding area is generally flat with the exception of concrete debris and aggregate mounds in the northern portion of the site. These mounds are moved by the batch plant personnel on an on-going basis; however, at the time of the survey, the mounds were at elevation +27 feet, and were located in the northeastern portion of the batch plant. Most of the batch plant area is between elevation +10 and + 13 feet. The middle section of the project site gently slopes to the north from about elevations +14 to +12 feet. The southern portion of the project site is at approximately elevation +14 to +15 feet.

Overhead utility lines span in a north-south direction along the eastern and western property lines. Additional overhead lines are present in the batch plant area near the trailers on the western edge of the site. No known underground utilities were noted during our subsurface investigation.

The area west of the project site, from Illinois Street to the proposed Louisiana Street (aligned approximately along the western property line of the project site) was reclaimed from the Bay by approximately 1935 (AGS, 1999). Review of historic aerial photographs indicates that by 1946 fill operations continued to extend the shoreline further east, creating the footprint on which the site currently lies. A later aerial photograph shows that the current general shoreline configuration, including Pier 80 and the truck trailer holding yard east of the project site, was built out by 1969.

Geotechnically challenging subsurface conditions exist at the project site. During our subsurface investigation we encountered abundant debris in the artificial fill consisting of concrete, metal, brick, and glass. It should be noted that drilling through the artificial fill was time consuming and drilling equipment was damaged on some concrete slabs and metal objects. Table A-2 – Debris Encountered During Subsurface Exploration in Appendix A summarizes notable debris encountered during our exploration and approximate time taken to drill though the fill.

SEISMICITY

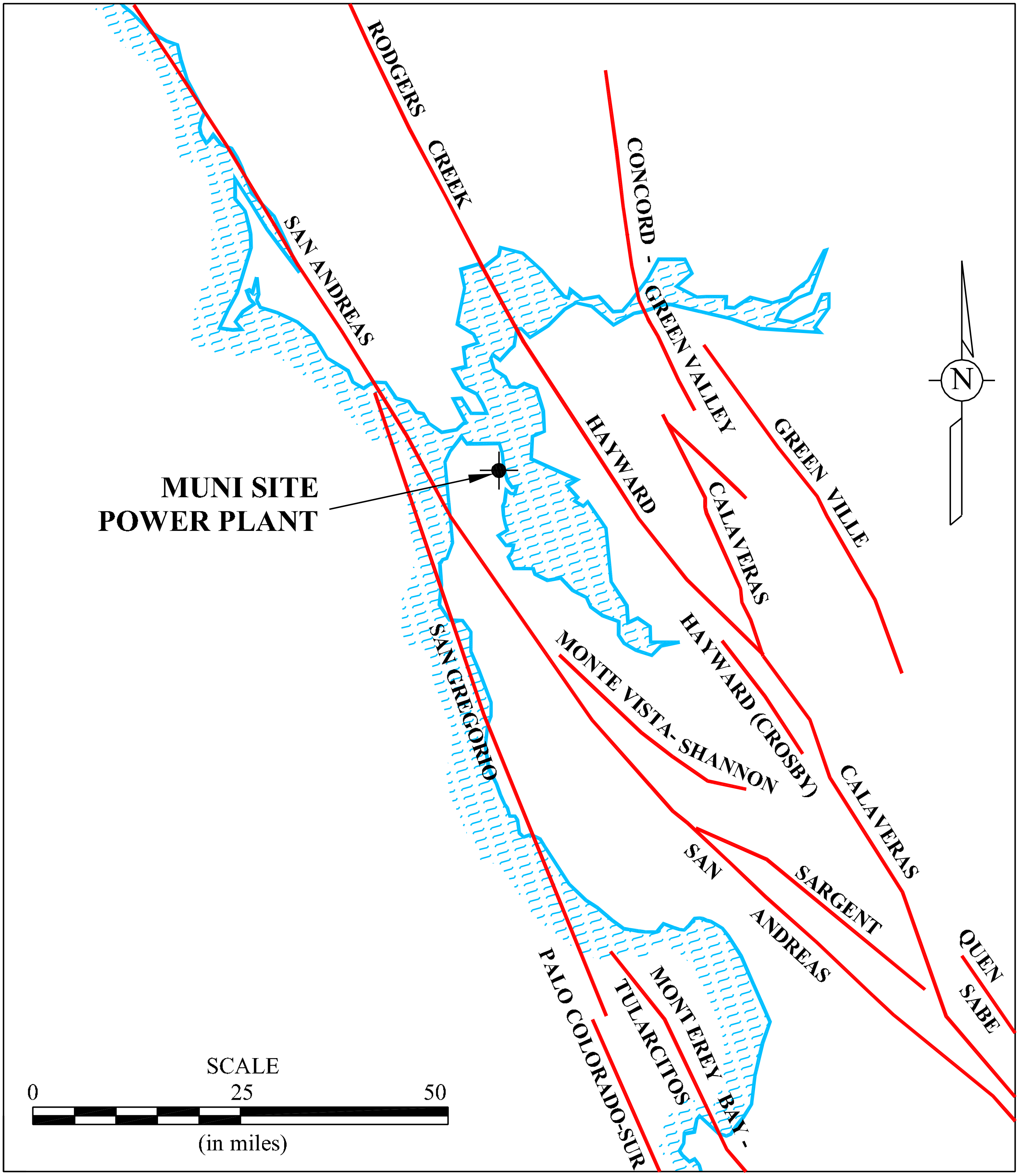
The San Francisco Bay Area contains several active faults that could cause strong ground shaking at the project site. Figure 2 – Regional Fault Map shows faults in the area and in relation to the proposed Muni Site Power Plant. Historic earthquake records compiled since 1800 indicate that five earthquakes of magnitude 6.0 or greater have occurred within 20 miles of the project site (Blake, 1993). The United States Geological Survey (USGS) Working Group on California Earthquake Probabilities concludes that there is a 62 percent probability of a strong earthquake ($Mw \geq 6.7$) occurring in the San Francisco Bay Region in a thirty year period between 2003 and 2032 (USGS, 2003). The major active faults in the project area comprise a complex system of right-lateral, strike-slip faults; including the San Andreas, San Gregorio, Hayward, and Calaveras faults; collectively known as the San Andreas fault system. The San Andreas, San Gregorio-Seal Cove, Hayward, and Calaveras faults have produced measurable historic ground motion and movement. Of these faults, the San Andreas is the controlling fault with respect to seismic design at the proposed power plant site. The California Geologic Survey (CGS), who recently updated fault parameters (Cao, et al, 2003), estimated that the San Andreas fault is capable of producing an earthquake of an estimated maximum moment magnitude of 7.9. A summary of nearby faults is presented in Table 2 - Active and Potentially Active Faults.

TABLE 2 – ACTIVE AND POTENTIALLY ACTIVE FAULTS

Fault (Segment or Event)	**Distance (miles)**	**Estimated Maximum Moment Magnitude**[1]	**Historic Earthquakes**[2]	
			Year	**Magnitude**
San Andreas			1838 1898 1906 1989	6.8 6.2 8.1 7.1
(1906 rupture)	8.2[3]	7.9[3]		
(Peninsula)	8.1	7.1		
(North Coast Segment)	10.7	7.4		
Hayward			1868	6.8
(South)	10.8	6.7		
(North)	11.0	6.4		
San Gregorio-Seal Cove			NA	NA
(North)	11.8	7.2		
Monte Vista-Shannon	24.0	6.7	NA	NA
Calaveras	20.6	6.8	1861 1979 1984	5.3 5.9 6.1

(1) Maximum Moment Magnitude based on CGS fault parameters updated in 2002 (CAO, et.al., 2003)
(2) Historic earthquakes shown may have occurred in other segments of the noted fault.
(3) 1906 rupture event assumes rupture of North Coast, Peninsula, and Santa Cruz Mtns. segments to San Juan Bautista. Maximum magnitude based on 1906 average 5 m displacement (WGCEP, 1990; Lienkaemper, 1996).

FIGURE 2
REGIONAL FAULT MAP



SOURCE: Blake, 2000-EQFault Computer Program.

We utilized the services of Dr. Robert Pyke to develop site-specific ground surface response spectra in accordance with FEMA 356, Section 1.6.2. The results of the site-specific analysis are included in his report, attached as Appendix B – Site Specific Dynamic Response Analysis.

GEOLOGY

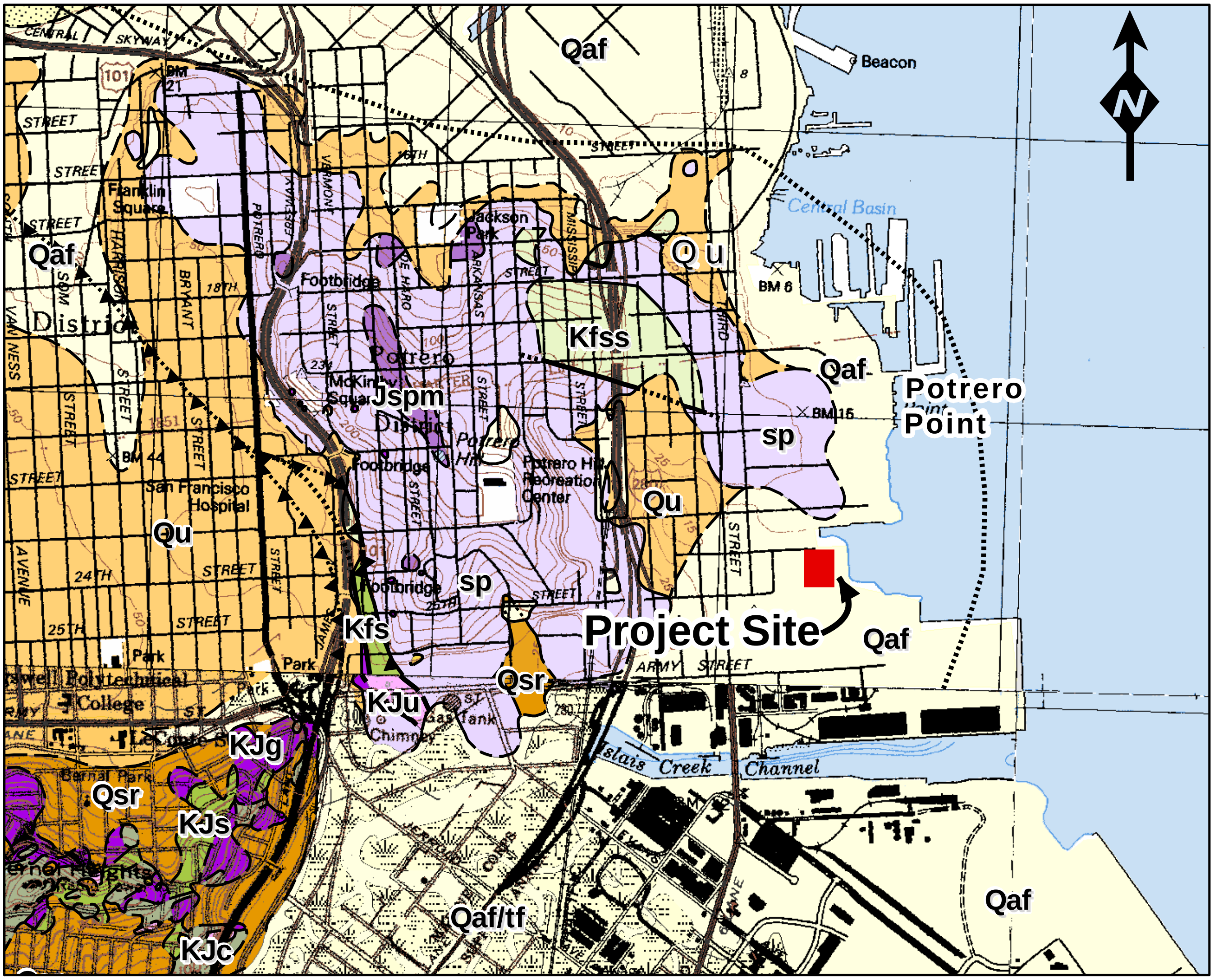
Regional Geology. The San Francisco Bay Area is located within the Coast Ranges Geomorphic Province. Past episodes of tectonism have folded and faulted the bedrock, creating the regional topography of northwest trending ridges and valleys characteristic of the Coast Range Geomorphic Province. Faults belonging to the San Andreas system have divided the bedrock underlying the San Francisco Bay Area into major structural blocks. The site is located on the San Francisco Bay Block which is bounded on the east by the Hayward fault and on the west by the San Andreas Fault. During the past two million years, the San Francisco Block has tilted to the east forming the elongated depression now occupied by the San Francisco Bay. During the same period, the Santa Cruz Mountains, Diablo Range, and Berkeley Hills have been uplifted.

The bedrock of the San Francisco Block consists of Jurassic-Cretaceous rock belonging to the Franciscan Complex and Great Valley Sequence. Rocks of these formations include graywacke sandstone, conglomerate, chert, serpentinite, shale, cataclasite, and altered volcanics.

Local Geology. The local geology is presented on Figure 3 – Local Geologic Map. Based on a review of published mapping (CDMG, 1969) and previous geotechnical reports in the project vicinity, bedrock in the project vicinity is overlain by approximately 170 feet of older bay mud, upper and lower layered sediments, younger bay mud, and artificial fill. Table 3 – Anticipated Project Site Subsurface Stratigraphy presents a schematic stratigraphic column of the different soil and rock types anticipated to underlie the proposed power plant site, and is based on information obtained from published mapping (CDMG, 1969) and previous geotechnical reports.

The older bay mud typically consists of medium-stiff to stiff, overconsolidated, plastic, fat clay with layers of dense silty clay to clayey sand, deposited under estuarine conditions and subsequently exposed by low sea level during periods of Pleistocene glaciation. The layered sediments, deposited during periods when older bay mud was deposited further off shore, is generally composed of silty and clayey sand and sandy clay. Layered sediments are subdivided into upper and lower units that are known to interfinger with the older and younger bay mud (URS, 2001).

FIGURE 3
GEOLOGIC MAP



Source: USGS Open File Report 98-354 and USGS MF-2337.

1,000 0 1,000 2,000 3,000 4,000 5,000 FEET

SCALE

LEGEND

Surficial Deposits

- Qaf - Artificial Fill
- Qaf/tf - Artificial Fill over Tidal Flat
- Qb, Qs - Beach Sand
- Qsr - Slope Debris and Ravine Fill
- Qu - Undifferentiated Surficial Deposits

Franciscan Complex

- Kfs, KJs - Sandstone and Shale
- Kfss - Massive sandstone
- KJfch, KJc - Chert
- KJg, Jfg, Jfgs - Greenstone
- Jspm - Massive Serpentinite
- KJu, fsr - Melange
- sp - Serpentinite

- contact, approx. located
- contact, certain
- contact, concealed
- fault, approx. located
- fault, certain
- fault, concealed
- thrust fault, certain
- thrust fault, concealed

The deposits of older bay mud and upper layered sediments are overlain by an approximately 20 to 30-foot thick layer of younger bay mud. The younger bay mud consists of soft, plastic, normally to slightly over-consolidated, lean to fat clay containing occasional lenses of organics. These geologically recent bay mud deposits are characterized by their high water content and compressibility, low dry density, and very low shear strength. The younger bay mud is overlain by a 25 to 30 foot thick layer of artificial fill comprised predominately of gravelly clay to clayey sand with gravel. Gravel consists of fragments of the Franciscan complex, rock, concrete, and brick. Other material in the debris fill includes metal, wood, organic material and evidence of oil or hydrocarbons.

TABLE 3 – ANTICIPATED PROJECT SITE SUBSURFACE STRATIGRAPHY

Geologic Era	Regional Classification	Approximate Elevation Range (ft)	Soil/Rock Types (Symbol)
Historic (0 to 200 years old)	Recent Fill	+14 to –17	Artificial Fill (af)
Holocene to Pleistocene (0 to 1.8 million years old)	Alluvial, Colluvial and Estuarine Deposits	-17 to –40	Younger Bay Mud (Qybm)
		-40 to –70, & -90 to –110	Upper Layered Sediments (Quls)
		-70 to –90, & -110 to –135	Older Bay Mud (Qobm)
		-135 to -158	Lower Layered Sediments (Qlls)
Cretaceous to Jurassic (65 to 165 million years old)	Franciscan Complex (KJ)	-158 and below	Sandstone (ss)
			Shale (s)
			Claystone (cs)

EARTH MATERIALS

Our exploratory borings preformed for this investigation, as well as review of previous borings by others in the immediate project vicinity, indicate that the site is blanketed by artificial fill underlain by younger bay mud, upper layered sediments, older bay mud, and lower layered sediments. These Quaternary deposits and sediments are underlain by Franciscan Complex bedrock at depths ranging from approximately 140 to 180 feet. Cross sections showing the general subsurface profile interpolated from our boring data are presented on Plate 3 - Subsurface Profile A-A' and Plate 4 – Subsurface Profile B-B' and C-C'. Locations of cross sections are delineated on Plate 2.

Artificial Fill (af). Deposits of artificial fill, which blanket the project site, were encountered in our exploration from the ground surface to depths ranging from 20 to 31 feet.

Review of historic aerial photographs suggests that this fill was placed at the site over a period spanning the early 1940's to the mid 1960's. Contours lines depicting the thickness of artificial fill are shown on Plate 5 – Thickness of Existing Artificial Fill. The fill predominantly consists of poorly graded to well-graded gravels (USCS classification symbol GP and GW) and silty to clayey gravels and sands (USCS symbols GM, GC, GW-GM, GP-GM, SM, and SP-SM). The gravelly soils typically contain approximately 50 to 80 percent gravel that is sub-rounded to sub-angular, with maximum dimension ranging from 1.5 to 3 inches; approximately 15 to 40 percent poorly graded sand; and, approximately 5 to 25 percent non-plastic silty to low plasticity clayey fines. The sandy fill soils typically contain approximately 45 to 55 percent poorly graded sand with approximately 25 to 45 percent fine to coarse gravel, and approximately 10 to 25 percent non-plastic silty to low plasticity clayey fines.

The fill is typically damp near the ground surface becoming moist to wet with depth. Measured dry density of the fill ranged from 80 to 132 pounds per cubic foot (pcf), with water contents ranging from 10 to 19 percent.

Near surface fill at the site consists generally of a thin layer of aggregates comprising gravel, sandy gravel, and crushed concrete. On the southernmost 1/3 of the project site, the surface aggregates are uniformly placed (6-inches to a foot) with an underlying geotextile (likely placed as a reference/separator for future reclaiming of the aggregate base). Beneath the near surface aggregates, the uppermost 5 to 8 feet of fill across the site consists of relatively dense to very dense Franciscan Complex derived soil containing serpentinite gravel-sized, and scattered cobble-sized, clasts in a matrix of silt and clay derived from decomposed serpentinite along with other sediments. This upper layer also contains minor debris including crushed brick and trace glass.

Below 5 to 8 feet, to depths of 20 to 31 feet, the composition of the fill consists of a highly heterogeneous composition of sandy and gravelly soils containing varying amounts of debris, which were at times very abundant to predominant. The debris encountered during our drilling primarily consists of concrete, asphalt, brick, wood, and metal. The debris, at times, imposed very difficult drilling conditions during our exploration (e.g., some debris required coring through with "trash barrel"), although all of our borings were successfully completed. A summary of specific debris and difficult drilling conditions encountered in our borings are presented in Appendix A in Table A-2 – Debris Encountered During Subsurface Exploration.

In addition to our boring exploration, the presence, location, and type of debris within the artificial fill were assessed by performing an electromagnetic and electrical resistivity survey of the site. The survey was performed by Southwest Geophysics, Inc., and the findings of their survey are presented in their geophysics report, Appendix C.

Younger Bay Mud (Qybm). Artificial fill at the project site is underlain by younger bay mud, which extends to about elevation -34 to -52, as shown in Plate 7 – Bottom

Elevation of Younger Bay Mud. The younger bay mud ranges from 19 to 35 feet in thickness, as shown on Plate 8 – Thickness of Younger Bay Mud. The younger bay mud generally consists of soft to medium stiff, highly compressible, dark greenish gray fat clay (USCS symbol "CH"). The younger bay mud typically includes zones with trace to abundant shell fragments, and trace to minor organic material. Occasionally, the younger bay mud possesses a mild H_2S odor.

Our laboratory testing indicates that the younger bay mud is slightly over- to slightly under-consolidated, which suggests that some minor consolidation of the younger bay mud is ongoing as a result of historic fill placement. Laboratory-measured dry density of the younger bay mud ranged from 65 to 74 pcf, with measured water contents ranging from 45 to 58 percent. Measured liquid limit (LL) ranges from 65 to 75, with a plasticity index ranging from 38 to 50. Laboratory measured unconsolidated, undrained (UU) shear strength of the younger bay mud ranges from 684 to 1481 pounds per square foot (psf). Additionally, field measurements of younger bay mud strength were made using pocket penetrometer and torvane instruments, which indicated undrained shear strengths ranging from 0.20 to 1.6 kips per square foot (ksf).

Upper Layered Sediments (Quls). A sequence of inter-bedded alluvial and marine sediments was encountered underlying the younger bay mud between elevations of –40 to –70 feet and underlying the upper layer of older bay mud between elevations of –90 to -110 feet. This unit is sometimes referred to as the San Antonio Formation (e.g., Rogers and Figuers, 1991), but has also been identified by numerous other names in past studies, including Older Alluvium (GTC, 1995 and 2005), Upper Layered Sediments (ADEC, 1999) and Upper Alluvial/Marine Sediments (URS, 2001). Throughout this report, we will refer to this unit as "Upper Layered Sediments" (Quls). Based on our boring observations, this unit consists of alternating layers of silty sands (SM), clayey sands (SC), sandy to clayey silts (ML), lean to fat clays (CL, CH), and clean poorly graded sands (SP). The alluvial deposits within the unit are typically yellowish brown to dusky yellow, while the marine sediments are typically grayish green and dark greenish gray.

The granular materials within this unit are typically fine to very fine grained sands that are dense to very dense. Laboratory-measured dry density of sandy soils within the unit ranged from 105 to 113 pcf, with water contents ranging from 18 to 21 percent.

The fine grained soils within this unit are typically stiff to very stiff. Field strength measurements of upper layered sediments were made using pocket penetrometer and torvane instruments, which indicated undrained shear strengths ranging from 1.2 to 4.75 kips per square foot (ksf).

Older Bay Mud (Qobm). Interfingered with the upper layered sediments is a relatively uniform layer of stiff fat clay (CH) commonly known as older bay mud. We encountered two layers of older bay mud, separated by a layer of upper layered sediments, in all of our deep borings at approximate elevation -70 to –90 for the upper layer and -110 to –135 feet for the lower layer.

The older bay mud encountered in our exploration predominantly consists of greenish gray to dark greenish gray fat clay (CH) that is moist and medium stiff to stiff with trace amount of shell fragments. Occasional lenses of clayey silt and lean silty to sandy clay were also encountered in our exploration. Laboratory measured shear strengths from unconfined compressive strength tests ranged from 1339 to 3722 psf. Additionally, field measurements of younger bay mud strength were made using pocket penetrometer and torvane instruments, which indicated undrained shear strengths ranging from 0.50 to 2.3 kips per square foot (ksf). Measured liquid limit (LL) ranges from 75 to 93, with a plasticity index ranging from 50 to 60. Laboratory-measured dry density ranged from 62 to 96 pcf, with water contents ranging from 28 to 65 percent.

Lower Layered Sediments (Qlls). A sequence of interbedded alluvial and marine sediments was encountered in B-4 underlying the older bay mud between elevation -135 to –158 feet. Similarly to the Upper Layered Sediments (Quls), this unit has been referred to by different names in past studies (e.g., GTC 1995, ADEC 1999, URS 2001, etc.); however, throughout this report, we will refer to this unit as "Lower Layered Sediments" (Qlls). Based on our observations in boring B-4, this unit consists of alternating layers of alluvial sandy clays (CL) and marine deposited fat clays (CH). The alluvial sandy clays typically contain moderate fine gravel derived from Franciscan complex shale. The sediments within the unit are typically grayish blue green.

Franciscan Complex (KJf). In boring B-4, bedrock of the Franciscan Complex was encountered at a depth of 168 feet, corresponding to elevation –157.5 feet. Published maps (CDMG, 1969) suggest that the bedrock elevation beneath the site ranges from very approximately –140 to –180 feet. The bedrock encountered in boring B-4 consists of dark gray to black, fractured, moderately strong shale.

GROUNDWATER

Groundwater was measured in borings that were initially cored with a "trash barrel" prior to changing to the rotary wash drilling method. We obtained groundwater measurements in all of the borings except B-12. A summary of all measured groundwater depths and corresponding elevations are presented in Table 4 – Groundwater Measurements. The groundwater level throughout the project site is likely to experience some tidal influence from the nearby San Francisco Bay and will likely fluctuate relative to daily high and low tide levels.

TABLE 4 – GROUNDWATER MEASUREMENTS

Boring	Date	Depth to Groundwater (feet)	Groundwater Elevation (feet, NAVD 1988 Datum)
B-1	7/23/05	10.5	1.5
B-2	7/30/05	9.4	4.1
B-3	7/23/05	10.2	3.3
B-4	7/23/05	10.0	0.5
B-5	7/27/05	9.4	1.6
B-6	7/27/05	9.3	1.7
B-7	8/2/05	11.5	0.0
B-8	7/28/05	12.3	-0.8
B-9	8/1/05	10.2	2.3
B-10	7/25/05	11.7	0.8
B-11	7/26/05	12.5	1.0
B-12	7/26/05	-	-
B-13	7/25/05	12.0	2.0
B-14	7/22/05	11.7	0.8
B-15	7/20/05	12.7	1.8

CONCLUSIONS AND RECOMMENDATIONS

1.0 FEASIBILITY

Based on our exploration, laboratory testing, and geotechnical analyses, it is considered geotechnically feasible to develop the power plant at the proposed site, provided that the findings and design considerations presented in this report are considered in the project design. Geotechnically challenging conditions requiring particular attention include:

- The amount and nature of the debris in the artificial fill,
- Weak and compressible bay mud underlying the proposed power plant,
- Potentially highly corrosive subsurface shoreline environment, and
- Potential for seismic hazards including strong ground shaking, liquefaction, and seismic settlement.

2.0 SEISMIC DESIGN CONSIDERATIONS

2.1 **General.** As the project site is located within the seismically active San Francisco Bay Area, major earthquakes can cause strong ground shaking and associated fault rupture, liquefaction, lateral spreading, landslides, and seismic settlement. These seismic hazards are discussed, and design considerations provided, in the following sections.

2.2 **Fault Rupture.** No active or potentially active faults are known to cross the site. Consequently, the hazard posed by ground rupture due to fault offset is considered to be very low, and does not warrant mitigation design considerations.

2.3 **Ground Shaking**. Although no known active faults traverse the site, strong ground shaking may occur as the result of a moderate to large earthquake occurring on one of the active regional faults. Of the active regional faults, the San Andreas fault is considered to be the most capable of causing strong ground shaking within the project site because of its estimated relative activity and proximity.

Dr. Robert Pyke assessed the site-specific ground shaking characterization of the project site which is provided in Appendix B and further discussed in Section 2.8. From his analysis, the peak horizontal ground acceleration (PGA) was calculated to be approximately 0.21g for Basic Safety Earthquake 1 (BSE-1) and 0.26g for Basic Safety Earthquake 2 (BSE-2) hazard levels. The relatively low PGA at the site (i.e., low with respect to standard attenuation relationships for soil and rock sites) is due to the anticipated large shear strains within the young bay mud during the design earthquake, thereby creating a large damping effect on high frequency ground motions. Conversely, longer period ground motions are relatively higher than typical rock or stiff soil sites due to the amplification effects of long period vibrations in the bay mud. For this reason, structures having long periods (e.g. tall or flexible structures) will likely experience high seismic forces and large displacements during a large earthquake. We recommend that the site-specific ground surface response spectra provided in Dr. Pyke's report, and Section 2.8, be used for design of power plant structures.

2.4 **Liquefaction**. Liquefaction is a phenomenon wherein a temporary, partial loss of shear strength occurs in a soil due to increases in pore pressure that result from cyclic loading during earthquakes. Saturated, loose to medium dense sands and silty sands are most susceptible to liquefaction, although documented field cases have shown that gravelly soils and certain fine grained soil are also capable of liquefying. Consequences of liquefaction can include ground settlements, foundation failure, sand boils, and lateral spreading.

Considering the high plasticity and fine-grained nature of the native bay mud, and the high density of underlying upper layered sediments, the potential for liquefaction of native soils beneath the upper areal fill at the proposed power plant is very low.

The soil layer most susceptible to liquefaction at the proposed power plant site is the artificial fill, which extends from the ground surface to a depth ranging from approximately 20 to 30 feet. We first assessed liquefaction potential of the fill layer with respect to soil type only, using recent methodology prescribed by Seed et al., 2003. The methodology improves upon the previous state of practice known as the Modified Chinese Criteria (developed by Wang, 1979; Seed and Idriss, 1982; and subsequently re-evaluated, modified, and transposed to U.S. conventions by Andrews and Martin, 2000). According to Seed et al., liquefaction susceptibility of silty and clayey granular soils is related to the fines content (-200 sieve), plasticity of the fines, and the natural water content of the soil.

As discussed under "Subsurface Conditions," the gravelly fill soils typically contain approximately 50 to 80 percent gravel that is either poorly or well graded, sub-rounded to sub-angular, with maximum dimension ranging from 1.5 to 3 inches; approximately 15 to 40 percent poorly graded sand; and, approximately 5 to 25 percent non-plastic silty to low plasticity clayey fines. The sandy fill soils typically contain approximately 45 to 55 percent poorly graded sand with approximately 25 to 45 percent fine to coarse gravel, and approximately 10 to 25 percent non-plastic silty to low plasticity clayey fines. Given these parameters, the areal fill at the project site, with the exception of some minor clayey zones and layers, falls into categories defined by Seed et al. (2003) as Zones A and B, which are delineated on an Atterberg Limit chart. Soils in these zones are considered susceptible to liquefaction provided that their water content is relatively high with respect to the liquid limit of the soil fines (i.e., w=80 to 85% LL). According to Seed et al. soils in Zone A are most susceptible to "classic" cyclically induced liquefaction, whereas soils in Zone B fall into a transition range, and in some cases, especially if the in-situ water content is greater than 0.85*LL, may liquefy, but tend to be more ductile and may not liquefy in the classic sense of losing a large fraction of their strength and stiffness at relatively low cyclic shear strains.

The next step of our liquefaction evaluation was to determine the soil susceptibility with respect to its in-situ state, specifically relative density indicated by Standard Penetration Test (SPT) blow counts. In general, for liquefaction of the site soils to occur, they must be saturated and their relative density, as indicated in the field by SPT blow counts or other means, should be high enough to resist seismically induced cyclic stresses (typically expressed in terms of the "cyclic stress ratio," or "CSR").

Based on our evaluation of soil resistance to CSR based on SPT blow counts (per Seed et al. 1985, with modifications by NCEER, 2001) we conclude that there is a potential for liquefaction to occur within the artificial fill at the site during a major earthquake. However, many of the zones of fill identified as potentially liquefiable are comprised of coarse gravelly soils, where pore pressures should dissipate rapidly, possibly limiting associated shear strains and strength loss within the soil.

Consequences of liquefaction of fill at the site include reduction (or loss) of soil bearing capacity, total and differential settlements, and ground fissures resulting from sand boils and lateral spreading. Because the upper 5 to 8 feet of the fill at the site is relatively dense to very dense, and lies above the groundwater table, we do not anticipate significant reduction of soil bearing capacity against shallow footings. A potential for lateral spreading may exist at the northeast corner of the site, which is discussed in Section 2.5. Seismic settlements are anticipated, resulting from post-liquefaction soil consolidation, and is further discussed in Section 2.6.

Typical schemes to mitigate the occurrence and/or effects of liquefaction include, but are not limited to, dynamic compaction, vibro-compaction or vibro-replacement stone columns, and in-situ grouting methods such as jet grouting and chemical grouting. However, because of the highly heterogeneous nature of the fill and abundance of debris within the fill at the site, we conclude that these mitigation measures would only provide marginal improvement to the liquefaction resistance of the soil, would be very difficult to implement and monitor their effectiveness, and would likely be cost prohibitive. We conclude that the most appropriate mitigation scheme is to support most, if not all, of the facility structures on deep pile foundations that derive their resistance from deeper dense soils.

2.5 **Lateral Spreading.** As the proposed power plant site is situated near and adjacent to the San Francisco Bay shoreline, the potential for lateral spreading during a major earthquake may exist, particularly at the northeast corner of the site, which is approximately 120 feet from the bay shoreline. According to Bartlett and Youd (1995), for significant lateral spreading displacements to occur, the soils should consist of saturated cohesionless sandy sediments with $N1_{(60)}$ less than 15, where liquefaction of the soils are likely based on standard liquefaction analysis. Soils in boring B-2 (i.e., northeast corner of the site) consist of loose cohesionless soils that are susceptible to liquefaction. However, the fill in boring B-2 is coarse gravel in nature with minor to moderate cobbles, which does not fall within the parameters applicable to the Bartlett and Youd lateral displacement model. We estimate that lateral displacements at the site will be possible but very small due to the gravelly nature of the fill, which typically does not undergo sustained loss of shear strength due to pore pressure increases, and hence, does not develop widespread shear zones for significant lateral displacements to occur during liquefaction. We estimate potential lateral movements at the northeast corner of the site on the order of a few (i.e., 1 to 2) inches during a major earthquake.

2.6 **Seismic Settlements.** Seismically induced settlement of on-site fill materials can occur in two manners: 1) settlements due to post-liquefaction volumetric reconsolidation of saturated soils, and 2) volumetric contraction ("densification") of non-saturated soils (above the water table) during strong ground shaking. We have estimated the magnitude of seismic settlement of artificial fill at the Muni Site for these two settlement modes based on the methodology prescribed by Tokimatsu and Seed (1987). Our estimates of seismically induced settlements are presented on Plate 9 - Seismically Induced Settlements.

As the Tokimatsu and Seed method is based primarily on the study of relatively clean saturated sands, some corrections were made to account for the fines-content of the fill materials, as well as the general gravelly consistency of the soil. Post-liquefaction SPT $N_{1(60)}$ corrections were made, as recommended by Seed (1987), to account for increased fines content. Based on our analysis, we estimate that seismically induced post-liquefaction settlement of saturated fill below the groundwater table range on the order of less than 1 inch to 5 inches. It is important to note that the fills at the site are very heterogeneous in nature, and consequently, the estimates of site settlements due to volumetric reconsolidation carry a high degree of uncertainty (studies of predicted vs. observed settlements [Wu, 2003] indicate an uncertainty factor of +/-2). The post liquefaction settlements should be considered as "averages," and local differential settlements should be expected.

Because of the generally very dense nature of the upper 5 to 10 feet of the artificial areal fill, as indicated by high SPT blow counts, our analysis indicates that seismic settlement due to densification of non-saturated granular soils above the groundwater table should be less than 1/2-inch across the site.

2.7 **CBC Seismic Design.** Because of the thickness and consistency of the soft, sensitive clays and potentially liquefiable fill present beneath the proposed power plant site, the Soil Profile Type for California Building Code Static Force Procedure design (CBC, 2001) is "$\mathbf{S_F}$." Soil Profile Type "$\mathbf{S_F}$" requires that a site-specific geotechnical investigation and dynamic response analysis be conducted to develop structural response spectra for proposed power plant elements for a given design seismic event. A discussion of the site specific response analysis, conducted by Robert Pyke, PhD, G.E., is provided in the following the following section.

2.8 **Site-Specific Dynamic Response Analysis.** We utilized the services of Dr. Robert Pyke to develop site-specific ground surface response spectra in accordance with FEMA 356, Section 1.6.2. The results of the site-specific analysis are included in his report, attached as Appendix B.

The ground surface response spectra were developed for two earthquake hazard levels, Basic Safety Earthquake-1 (BSE-1) and Basic Safety Earthquake-2 (BSE-2). As indicated in Dr. Pyke's report and in FEMA 356, the site-specific response acceleration parameters used in constructing the design spectrum are as follows:

- For BSE-2, the acceleration parameters are taken as the smaller of the values derived from: (1) the spectrum from 2 percent in 50 years probability of exceedance and (2) 150 percent of the mean deterministic spectrum.
- For BSE-1, the acceleration parameters are taken as the smaller of the values derived from: (1) the spectrum from 10 percent in 50 years probability of exceedance and (2) two-thirds of the BSE-2 spectrum.

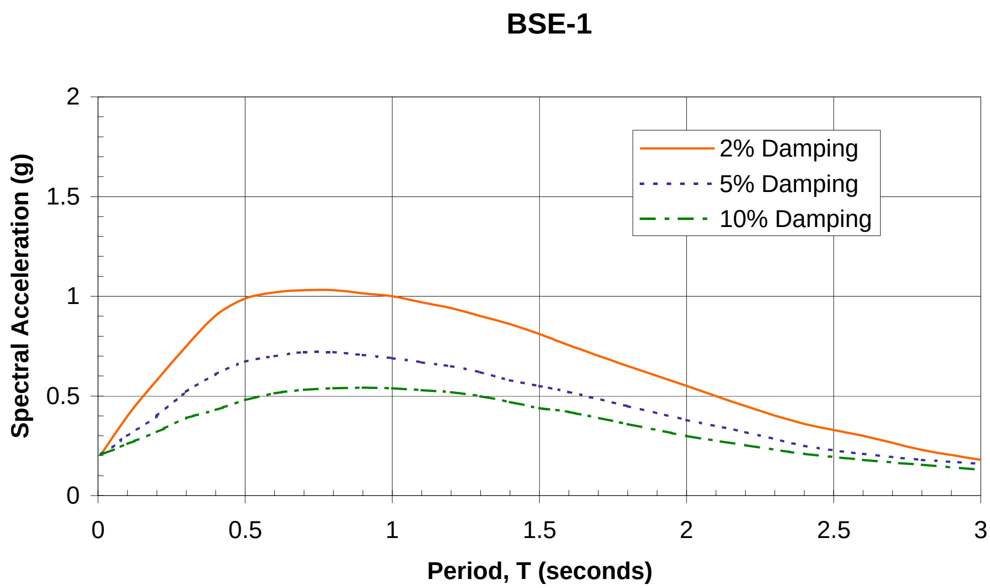
Once the site-specific response acceleration parameters were selected, Dr. Pyke performed one-dimensional nonlinear site response analyses in order to obtain horizontal motions at the ground surface. The input motions for the nonlinear analyses were in the form of acceleration time histories at the bedrock surface at a depth of 170 feet below ground surface. The spectra of the input motions are labeled MUNI BSE-1 "ROCK" in Figures 4 through 6 and MUNI BSE-2 "ROCK" in Figures 7 through 9 of Dr. Pyke's report (Appendix B). In comparing the input spectra to the computed ground surface spectra in Figures 4 through 9 of Dr. Pyke's report, the effect of the soil strata above the bedrock is to dampen the high frequency (low period) ground motions, and amplify the longer period ground motions. Dr. Pyke indicates that "the analyses for both the BSE-1 and BSE-2 levels of loading showed pronounced nonlinearity as a result of the fill serving as an inertial reaction that generates large shear strains in the young Bay Mud. As a result the ground surface motions that are obtained for BSE-2 are not much greater than those for BSE-1." It is this nonlinearity that acts to lessen the spectral accelerations at the ground surface at high frequencies.

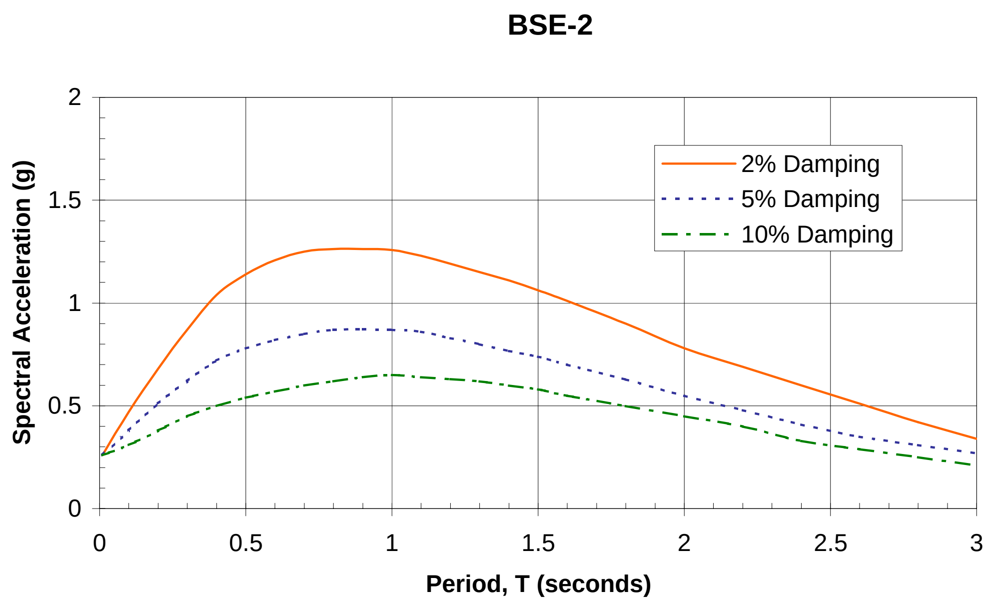
The recommended site-specific ground surface response spectra are provided in Dr. Pyke's report as Figure 10 for BSE-1 and Figure 11 for BSE-2. The response spectra are duplicated in this report on Figure 4 – Ground Surface Spectra.

3.0 GROUNDWATER

The measured groundwater in the upper artificial fill in our borings was measured at elevations between –0.8 and +4.1 feet. Due to the proximity of the San Francisco Bay, groundwater in the fill is likely hydraulically connected to the bay and levels will fluctuate with the changing tide. During the rainy season groundwater levels may be governed by rainwater infiltration, both on the site and at upgradient locations. Therefore, seasonably higher groundwater levels should be anticipated. Depending on the tide and/or seasonal conditions, we anticipate that groundwater will enter excavations approaching elevation as high as +6, or approximately 8 feet deep through the relatively

Figure 4
Ground Surface Spectra*





* Based on site-specific seismic hazard analyses performed by Dr. Robert Pyke (attached as Appendix B)

permeable fill material at the project site. The underlying bay mud is saturated, relatively impermeable, and difficult to drain. Excavations approaching or exceeding the groundwater level may necessitate dewatering during construction. The choice of a suitable dewatering scheme, its design, and implementation should be the responsibility of the contractor. The dewatering scheme chosen, designed, and implemented should consider the following objectives:

- Lower the groundwater table and intercept seepage which would otherwise emerge from the sidewalls or the bottom of the excavation;
- Improve the stability of the excavation at the sidewalls and the bottom; and
- Provide a reasonably dry work area in the bottom of the excavation throughout the backfilling operation.

The design of the dewatering system should include provisions for the collection and disposal of the water. Proper disposal will depend upon the nature and extent of the groundwater contamination, if any. Characterization of such groundwater contamination was not part of this study.

4.0 EARTHWORK CONSIDERATIONS

4.1 **General.** Given the earth materials on the project site encountered during our exploration, the contractor should be able to carry out planned excavations using conventional heavy equipment. However, we encountered hard drilling in several of our exploratory borings as summarized in the Earth Materials section and in Table A-2. Obstructions from debris such as old concrete and other rubble/debris should be anticipated during excavation. General geotechnical considerations for sub-grade preparation, excavations, bottom stability, general fill, engineered fill, engineered fill placement and compaction, and structural backfill are presented in the following sections.

Evaluation of the presence, or absence, and treatment of hazardous materials were not part of this study. If hazardous materials are encountered during excavation, proper handling and treatment during construction will depend on the contaminant type, concentration, and volatility.

4.2 **Site Preparation and Grading**. Site preparation will consist of excavation and removal of on-site materials such as pavement, concrete, fences, and miscellaneous debris in preparation of the foundation excavations. Also as part of site preparation, the location of underground utilities should be determined and, if affected by construction activities, should be relocated or protected. As described in the Site Conditions section, the batch plant activities of Pacific Cement has resulted in the site being generally covered with concrete rubble and concrete pads that serve as supports for construction trailers and other equipment. The site should be proof-rolled after excavation or backfilling operations are completed.

We understand that the project plans call for some nominal grading, including some cut of about 2 feet and about 0.5 to 3.5 feet of fill placement at some locations. Grading information available to us at this time is shown on Plate 6 – Thickness Contours of Proposed Cut and Fill. This grading is necessary to achieve proper site drainage. The placement of areal fill to achieve final design surface grades may induce some settlements of underlying soft compressible bay mud. For example, the placement of 3 to 3.5 feet of areal fill may cause up to 2 inches of consolidation settlement within the underlying younger bay mud. Additionally, based on our sampling, testing, and analysis, it is apparent that the younger bay mud may be undergoing some continued consolidation settlement resulting from loads imposed by the existing fill at the site, some of which was placed as recently as the late 1960's. Settlement resulting from consolidation of younger bay mud generally occurs gradually over a long time period, and is a function of bay mud properties, thickness of the bay mud layer, and drainage properties of the overlying and underlying soil strata. Table 5 – Estimated Time Rate of Consolidation, provides an estimate of the rate of consolidation (and hence, degree of ultimate settlement) with respect to the thickness of the younger bay mud.

TABLE 5 - ESTIMATED TIME RATE OF CONSOLIDATION

Time (years)	Average Degree of Consolidation (%)		
	H = 20 feet	H = 30 feet	H = 40 feet
1	45	33	25
2	70	45	35
5	90	75	55
10	95	90	75
25	100	99	95
50	-	100	100

Notes: 1 - H = Thickness of younger bay mud
2 - Based on coefficient of consolidation of 20 ft^2/year and double drainage conditions.

In addition to these primary consolidation settlements, the younger bay mud will likely continue to undergo secondary consolidation settlement for a period of several years after completion of primary settlement. Our estimates of anticipated settlements from primary consolidation of younger bay mud resulting from the existing fill and proposed new fill, as well as secondary consolidation settlement of younger bay mud, are shown on Plate 10 - Settlement Induced by Existing and New Fill. Design of grading and drainage should consider the estimated settlements shown on Plate 10.

4.3 **Excavations.** Excavations are expected to encounter very dense to hard gravelly fill, followed by gravelly, loose to medium dense fill material containing varying amounts of debris including concrete, brick, wood, and metal fragments. Shallow excavations for the power plant structures will allow for unshored excavations with adequately sloped sidewalls (within the upper areal fill), or vertical walled shored or braced excavations where space constraints may not allow for open, sloped excavations. At a minimum, excavations should be constructed in accordance with the current California Occupational Safety and Health Administration (OSHA) regulations (Title 8, California Code of Regulations) pertaining to excavations. Temporary cut slopes for shallow excavations within the fill are expected to be stable for configurations described in Title 8 for Type C soils and should be cut back no steeper than 1.5 horizontal to 1 vertical (1.5:1). All excavations should be closely monitored during construction to detect any evidence of instability.

Care should be taken when excavating near existing utilities and pipelines. New excavations can undermine support of adjacent existing pipelines and other subsurface structures. We recommend that some form of vertical shoring system should be considered for excavated sidewalls that are adjacent to existing pipelines or other known buried adjacent structures.

Particular attention should also be given to excavations near other existing structures, such as the existing warehouse along the southern boundary of the project site and structures associated with the proposed Muni Light Rail Vehicle Maintenance and Operation Facility, whose construction will likely be completed prior to construction of the power plant. Care should be taken to minimize prolonged lowering of groundwater below adjacent structures. If prolonged excavation dewatering is anticipated adjacent to existing structures, a system for monitoring effects of the excavation and dewatering should be established.

4.4 **General Fill.** On-site material that is determined non-hazardous and that is free of debris and other unsuitable materials may be used as general fill. Excavation and redistribution of the debris fill materials will likely require monitoring and screening as necessary. Any zones containing excessive debris should be identified, segregated from the suitable material, and disposed of appropriately. Areas receiving general fill should be limited to general grading, landscaping, and for areas that are not supporting structures. Typically, soils used as general fill should have a low potential for expansion (i.e., plasticity index less than 15 and liquid limit less than 40), and should be relatively free of organic matter and other unsuitable material; and rocks, broken concrete, or other solid materials greater than four inches in greatest dimension. Some fragments greater than 4 inches may also be incorporated into the fill provided that they are distributed in a manner that prevents nesting and so that the voids between large fragments are filled with finer material.

4.5 **Engineered Fill.** Placement of engineered fill may be needed to replace over-excavated soft soils or unsuitable full in preparation for construction of footings, slabs, or mats. Material for engineered fill should be non-hazardous, inorganic, well graded, free of rocks or clods greater than 4 inches in greatest dimension, and have a low potential for expansion. The material should have a liquid limit less than 35, a plasticity index less than 15 and no more than 25 percent passing the No. 200 sieve. Because the on-site fill contain a variety of debris, it is likely that it may be uneconomical to derive material suitable for engineered fill from on-site excavated materials. On-site material that is determined non-hazardous and that is free of debris and other unsuitable materials may be used as general fill for areas not supporting structures such as foundations, slabs, etc.

If large over-excavations are required to remove unsuitable subsoils for foundation preparation, consideration should be given to potential settlements of younger bay mud resulting from the net load increase associated with the placement of engineered fill in such over-excavations.

4.6 Engineered Fill Placement and Compaction. Engineered fill should be placed in layers no greater than 8 inches in uncompacted thickness, conditioned with water or allowed to dry to achieve a water-content near or slightly above optimum, then mechanically compacted to at least 90 percent relative compaction based on ASTM D1557. All engineered fill placed to support footings and the upper 6 inches of engineered fill supporting slabs-on-grade should be mechanically compacted to at least 95 percent relative compaction as determined by ASTM D1557. All compaction should be performed using mechanical compaction means; flooding or jetting should not be used as a means to achieve compaction. The ASTM D1557 laboratory compaction tests should be performed at the time of construction to provide a proper basis for compaction control.

4.7 Structural Backfill. Structures extending below grade should be backfilled with structural fill to a minimum width of two feet beyond the foundation footprint. Structural backfill should meet the following gradation:

Sieve Size	Percent Passing
3 inches	100
11/2 inch	80 to 100
#4	50 to 100
#16	40 to 90
#50	10 to 60
#200	0 to 10

Backfill should be moisture conditioned to within two percent above optimum, placed in layers not exceeding 8 inches in uncompacted thickness, and mechanically compacted to 90 percent relative compaction per ASTM D1557.

4.8 **Pipe Bedding.** Small diameter pipes and other utility lines servicing the power plant are anticipated. Unless concrete bedding is required around utility lines, pipe bedding placed in dewatered trenches should consist of well-graded sand or sand-gravel mixture. Maximum gravel size should be 0.5 inches and the bedding material should have less than 12 percent passing the No. 200 sieve. Uniformly graded material such as pea gravel should not be used as pipe bedding material. Pipe bedding should have a minimum thickness of 6 inches beneath the pipe and 6 inches above the pipe. If soft or otherwise unsuitable soils are exposed in the bottom of the trench excavation, the necessity of over-excavation should be evaluated by the project geotechnical engineer. All pipe bedding should be placed to achieve uniform contact with the pipe and a minimum relative compaction of 90 percent per ASTM D1557. Compaction of the pipe bedding by means of jetting or flooding should not be allowed.

4.9 **Utility Trench / Pipe Backfill.** Utility and pipe trenches may be backfilled above the pipe zone with excavated on-site soils, provided they meet the gradation requirements of engineered fill. The backfill material should be placed in layers no greater than 8 inches in un-compacted thickness, conditioned with water or allowed to dry to achieve a moisture-content near or slightly above optimum, then mechanically compacted to at least 90 percent relative compaction based on ASTM D1557. The upper 2 feet should be compacted to at least 95 percent relative compaction in areas where structural or traffic loads are anticipated.

5.0 FOUNDATION RECOMMENDATIONS

5.1 **General.** The Muni power plant site is underlain by several feet of uncontrolled artificial fill and soft younger bay mud sediments. These units pose several geotechnical challenges, as evidenced from our exploratory drilling as well as from the geophysical exploration. Based on our understanding of the proposed structures and the anticipated loads as provided to us by the designers, we conclude that the structures should be founded on deep foundations. Only lightly loaded structures that can withstand settlements with respect to pile-supported structures and that can be easily repaired by means of on-going maintenance in the event of excessive total and differential settlements may be supported on shallow footings.

We carried out a cursory evaluation of the suitability of ground modification techniques in lieu of deep foundations. Based on the support requirements of the proposed structures and based on the random character of the artificial fill, we concluded that ground modification methods are not suited for the project as described, as they may be cost prohibitive. In addition, the main difficulty with the project site lies with the randomness of the artificial fill and the likely obstacles that may be encountered during drilling, and most ground modification methods, other than deep dynamic compaction, require drilling.

Due to the stratigraphy of the subsurface materials underlying the site and due to the presence of high groundwater, pre-cast, pre-stressed concrete driven piles are the most suitable type of deep foundations for the project. Steel H-piles are also suitable for foundation support, but we are not recommending them as they are likely to be uneconomical for this project.

5.2 **Driven Concrete Piles.** Most of the proposed power plant structures, especially the heavily loaded structures such as water storage tanks, combustion turbine generators, SCR/CO catalyst systems, and stacks, should be supported on driven pile foundations. We recommend 14-inch square pre-cast, pre-stressed concrete driven piles that would derive their support predominantly through skin friction within the upper layered sediments (Quls) and older bay mud (Qobm) that is generally present below the younger bay mud (Qybm).

Such 14-inch square concrete piles appear to be the most efficient for the site, given the thickness of fill and the younger bay mud. For pile design, we have identified two distinct areas within the project site: Area B that is present in the middle of the site and two Area A's that are present on either sides of Area B. These areas are as shown on Plate 11 – Pile Design Areas A and B.

The pile tip elevations recommended herein correspond to allowable downward compressive capacities of 100 tons and 125 tons. Lower capacity piles are expected to be inefficient due to the 50- to 60-plus feet of artificial fill and younger bay mud that do not contribute to pile support. In the following sections, we denote the piles with allowable downward compressive capacities of 100 tons as "100-ton piles" and those with 125 tons allowable downward compressive capacities as "125-ton piles".

For structures to be supported within Area A, we recommend the following pile tip elevations: (1) Elevation -75 feet for the "100-ton" piles, and (2) Elevation -95 feet for the "125-ton" piles.

For structures to be supported within Area B, we recommend the following pile tip elevations: (1) Elevation -95 feet for the "100-ton" piles, and (2) Elevation -105 feet for the "125-ton" piles.

5.3 **Pile Lateral Capacity.** Resistance to lateral loading will be provided by passive resistance of the soil against the pile shafts. The lateral load response for 14-inch pre-stressed pre-cast concrete piles under "free head" and "fixed head" conditions was analyzed using load-deflection "p-y" analysis using the computer program LPILE (Reese and Wong, 1989). The "p-y" analysis models the non-linear pile-soil interaction along the depth of the pile. The estimated pile head deflection, the maximum moment in the pile, and the location of the maximum were computed for various lateral loads. The loads used in the analysis are actual loads and have not been modified to include any load factors. Results of the analysis for individual piles are presented on Figure 5 - Lateral Load versus Pile Head Deflection, and on Figure 6 - Maximum Pile Moment versus Pile Depth.

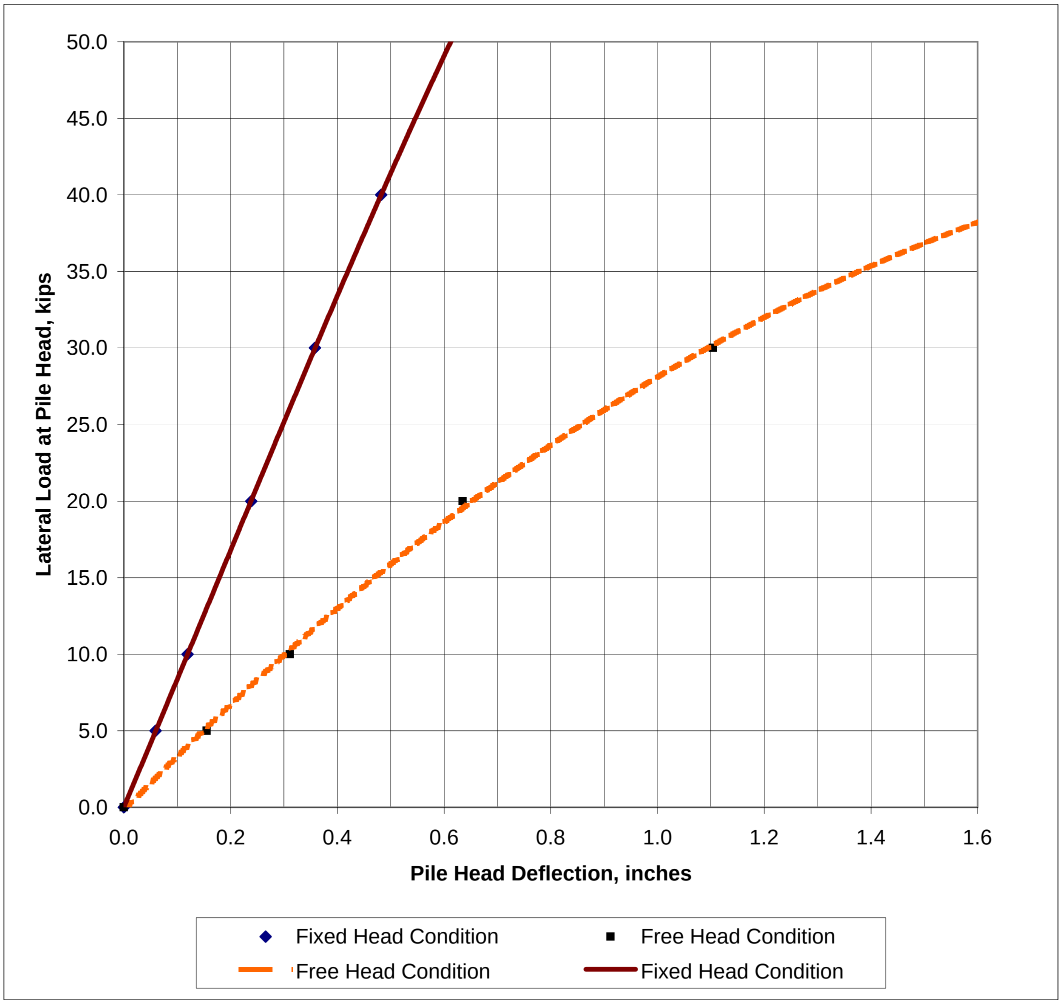
Figure 6 indicates that maximum moments occur in the pile at a depth between 5 and 10 feet, depending on the pile head condition ("free" or "fixed") and the imposed lateral load, and that moments decrease to zero at approximately 30 feet below the pile head. However, as discussed in the site specific dynamic response analysis (Appendix B), the potential exists during seismic activity for relatively large soil shear strains to occur at the interface of bay mud (younger and older) with upper layered sediments and artificial fill. These potential strains may induce significant shear and moment loads on the pile at depths greater than 30 feet. Consequently, we recommend that pile steel reinforcement designed for the loads, moments, and deflections shown on Figures 5 and 6 be extended the full length of the pile.

The results presented on Figures 5 and 6 are applicable for individual piles only. Lateral response of piles in a group is affected by pile spacing, pile orientation, and direction of loading. Hence, lateral resistance of pile groups can only be evaluated after the pile layout is determined. For preliminary design, in the case of piles in a row where the spacing is 3 pile diameters and the loading is in the direction of the pile row, it can be assumed that the lead pile in the row develops it full lateral load capacity, while the trailing piles develop 40% of their full lateral capacity. If the spacing is increased to 5 pile diameters, the piles may be assumed to develop their full capacity for the same direction of loading. For spacing between 3 and 5 pile diameters, the trailing piles capacity may be obtained by interpolation.

5.4 Pile Installation. Pre-drilling through the artificial fill will be required prior to installing the piles. The construction contractor should be prepared to encounter obstacles such as timber, concrete rubble, and other obstructions during pre-drilling. The findings of this report, including difficulties encountered during our exploratory drilling program, and the geophysics report presented in Appendix C, should provide some indication with regards to likely obstacles within the fill.

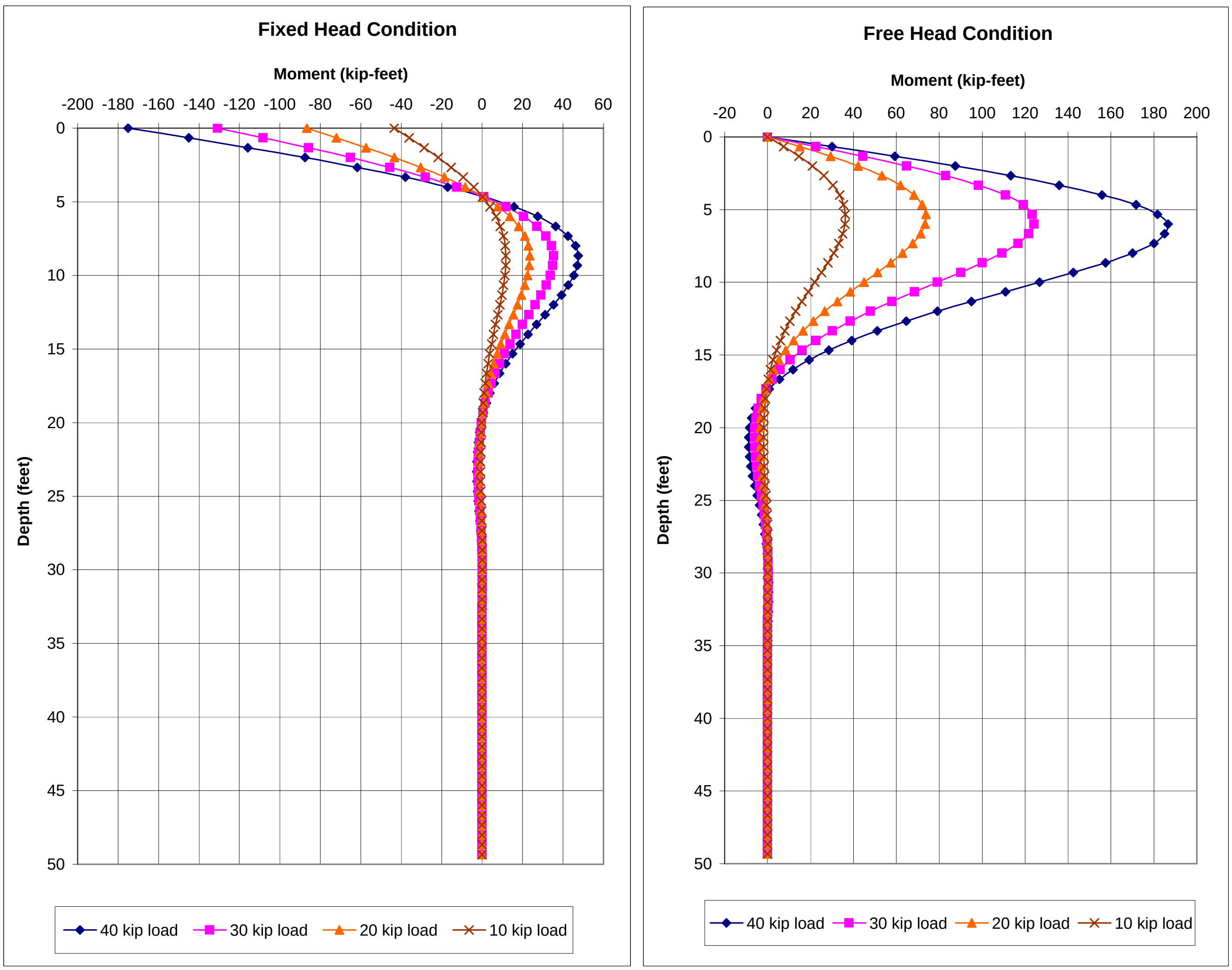
The 14-inch concrete piles should be installed using an appropriate pile driving system, such as a Delmag D46-32 diesel hammer, that is capable of delivering the necessary driving energy. These piles, if installed using a suitable hammer, are unlikely to encounter 'refusals' within the bearing strata. Depending upon the final foundation plans that are developed by the designers, it is likely that the piles may need to be driven using a follower.

FIGURE 5
LATERAL LOAD versus PILE HEAD DEFLECTION
Muni Site Power Plant



NOTE: Deflections shown are for 14-inch square pre-stressed pre-cast concrete piles only.

FIGURE 6
MAXIMUM PILE MOMENT versus PILE DEPTH
Muni Site Power Plant



NOTE: Moments shown are for 14-inch square pre-stressed pre-cast concrete piles only.

Upon completion of the design and hiring a contractor and prior to pile driving, we should be given an opportunity to perform a wave-equation type dynamic 'drivability' analysis of the contractor's proposed driving equipment to evaluate the adequacy of the driving system and to develop suitable pile driving criteria such as blow counts at the end of driving.

5.5 **Indicator Pile program/PDA Tests.** To better evaluate the driving characteristics of the pile-hammer system, to assess the feasibility and efficiency of using the contractor's preferred hammer, and to develop driving criteria (blows per foot, hammer type, fuel setting, etc.) for the production piles, we recommend that an indicator pile program be carried out prior to casting and delivering the production piles. Indicator piles are not 'sacrificial' elements and will function as part of the foundation system. We also recommend that some of the indicator piles be instrumented for PDA (Pile Dynamic Analyzer) testing and such PDA tests be performed on-site during installation of the indicator piles. Such PDA tests will be used to evaluate the driving stresses within the pile, to confirm the pile's integrity, as well as used to assess the as-driven design capacities of the piles. The number and location of the indicator piles should be developed once the foundation design is completed and the pile layout is available.

5.6 **Uplift.** Short-duration uplift forces imposed on piles due to overturning moments will be resisted by the buoyant weight of the piles and by the friction mobilized along the pile shaft. For calculating the uplift capacity required to resist short-term uplift forces, an ultimate uplift resistance of 100 tons per pile may be considered for the "100-ton" piles and an ultimate uplift resistance of 125 tons for the "125-ton" piles.

5.7 **Downdrag.** It is likely that some amount of downdrag loads due to negative skin friction within the artificial fill and younger bay mud are imposed on the piles. We have taken into consideration the likelihood of such downdrag forces in developing our recommendations for the minimum pile tip elevations.

5.8 **Shallow Foundations.** The designers should consider using pile foundations for most of the power plant structures. However, lightly loaded structures that can undergo relatively large differential settlements, especially with respect to other pile supported structures, may be supported on shallow footings. Such structures should also not require excavations deeper than about 2 to 3 feet below existing grade, as such excavations may encounter significant quantities of unsuitable materials. The maximum width of shallow footings, such as footings for walls and lightly loaded columns, should not exceed 3 feet.

Foundations that are constructed per the requirements presented in Section 4 - Earthwork should have allowable bearing capacities of 2,000 pounds per square foot (psf) to resist dead plus normal duration live loads. The recommended allowable bearing capacity may be increased by one-third for short-duration wind and seismic loads. The allowable bearing capacity value presented here has a factor of safety of at least 3.0 against bearing failure.

5.9 **Foundation Settlements.** Structures that are founded on 14-inch pre-cast pre-stressed concrete piles as recommended in this report may settle less than 0.5 inches during or immediately after construction. Long-term settlements of the pile foundations are unlikely. Shallow footings, designed and constructed in accordance with the recommendations of Section 5.7, should not induce any further consolidation of the younger bay mud, but should be expected to undergo total settlements on the order of 0.5 to 1.5 inches due to loads imposed by existing and proposed new areal fill. Due to likely on-going consolidation settlement of the younger bay mud and due to the randomness of the artificial fill, differential settlements with respect to pile supported structures are likely to be significant over time, on the order of 4 to 8 inches, resulting from a combination of consolidation settlement of the younger bay mud and potential seismic settlements of site fill. A breakdown of anticipated settlement modes and magnitudes for shallow and deep foundations is shown on Table 6 – Settlement Estimates. Refer to Plates 9 and 10 for estimates of seismic induced settlement of artificial fill and consolidation settlement of younger bay mud, respectively. Elements of project design, such as utility lines, should be designed to accommodate such differential settlements.

TABLE 6 – SETTLEMENT ESTIMATES

Settlement Mode	Anticipated Settlement (inches)	
	Shallow Footing Foundations	Deep Pile Foundations
Post Construction Immediate Settlement	< ½	< ½
Liquefaction/Seismically Induced Subsidence	< 1½ to 5½	NA[1]
Consolidation Settlement	¼ to 3	NA
Secondary Consolidation Settlement	1½ to 2	NA

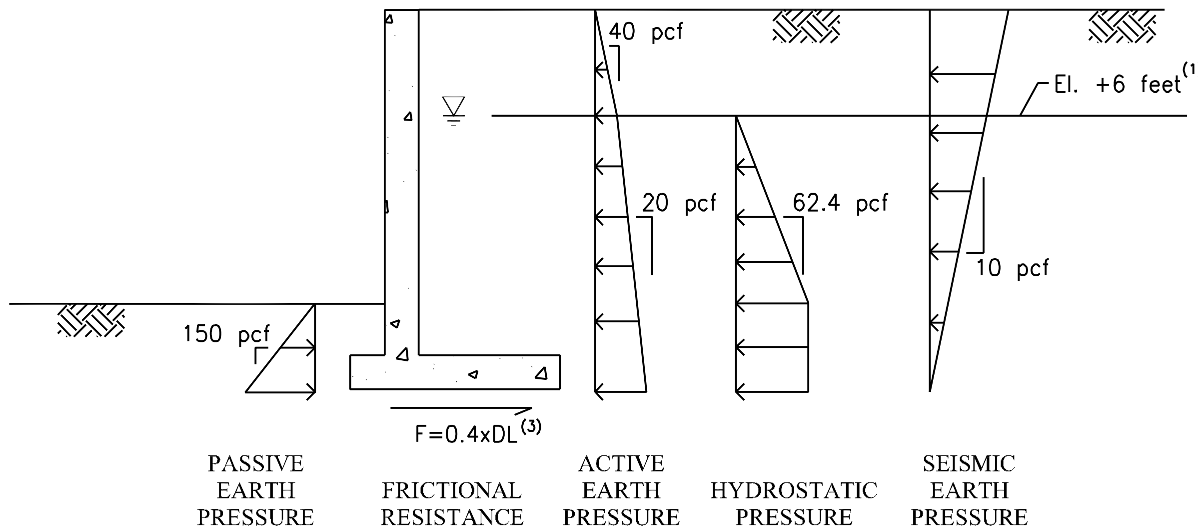
[1] NA: Not Anticipated

6.0 LATERAL EARTH PRESSURES

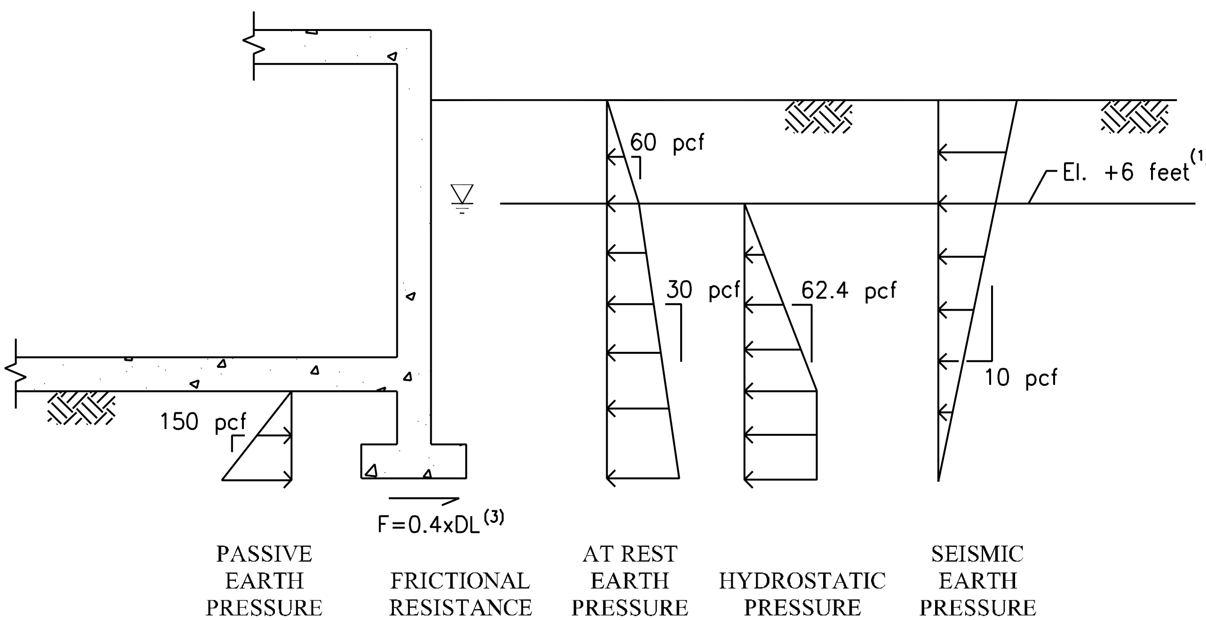
Structural components that extend below ground surface, such as foundations, below-grade walls, and temporary shoring systems, will experience lateral earth pressure from the soil and hydrostatic pressure from any existing groundwater. Recommendations for design and assessment of the active, at-rest, passive, and seismic earth pressures, and coefficient of base friction to resist active and at-rest loads for restrained and unrestrained walls are provided in Figure 7 - Lateral Earth Pressures for native areal fill material and Engineered Fill. Active earth pressures are imposed by the soil on walls that are free to translate or rotate at least 0.004H, where H is the height of the wall. If walls are restrained or deflections are undesirable, at-rest earth pressures should be used in design.

FIGURE 7
LATERAL EARTH PRESSURES

UNRESTRAINED WALLS



RESTRAINED WALLS



NOTES:

(1) ASSUMES WALLS ARE NOT DRAINED SO THAT HYDROSTATIC PRESSURES BUILD UP TO THE DESIGN GROUNDWATER ELEVATION OF +6 FEET (NAVD 1988).

(2) SURCHARGE PRESSURES SHOULD BE ADDED WHERE APPROPRIATE AS EVALUATED BY THE STRUCTURAL ENGINEER.

(3) DL = DEAD LOAD ON BASE OF FOOTING.

Loads on walls and structures can be resisted by passive pressures that develop against the side of the below-grade structure. Surcharge loading from adjacent structures should be evaluated separately. The passive EFP shown on Figure 7 has been reduced from the ultimate passive resistance to limit lateral deflections.

7.0 UPLIFT RESISTANCE

Structural components that extend below ground surface, if not designed with an extensive drainage and permanent dewatering system, will experience hydrostatic uplift pressure. Recommendations for design and assessment of uplift resistance of below-grade structures are provided in Figure 8 – Uplift Resistance. Additional uplift resistance may be provided by utilizing driven concrete piles. Uplift capacities of piles are provided in the Foundation Recommendations section of this report.

8.0 WATERPROOFING

Waterproofing is often a critical element in protecting the use of structures that extend below the groundwater table. As such, an engineer or architect familiar with the design and installation of waterproofing systems for slabs and walls below the groundwater level should be consulted.

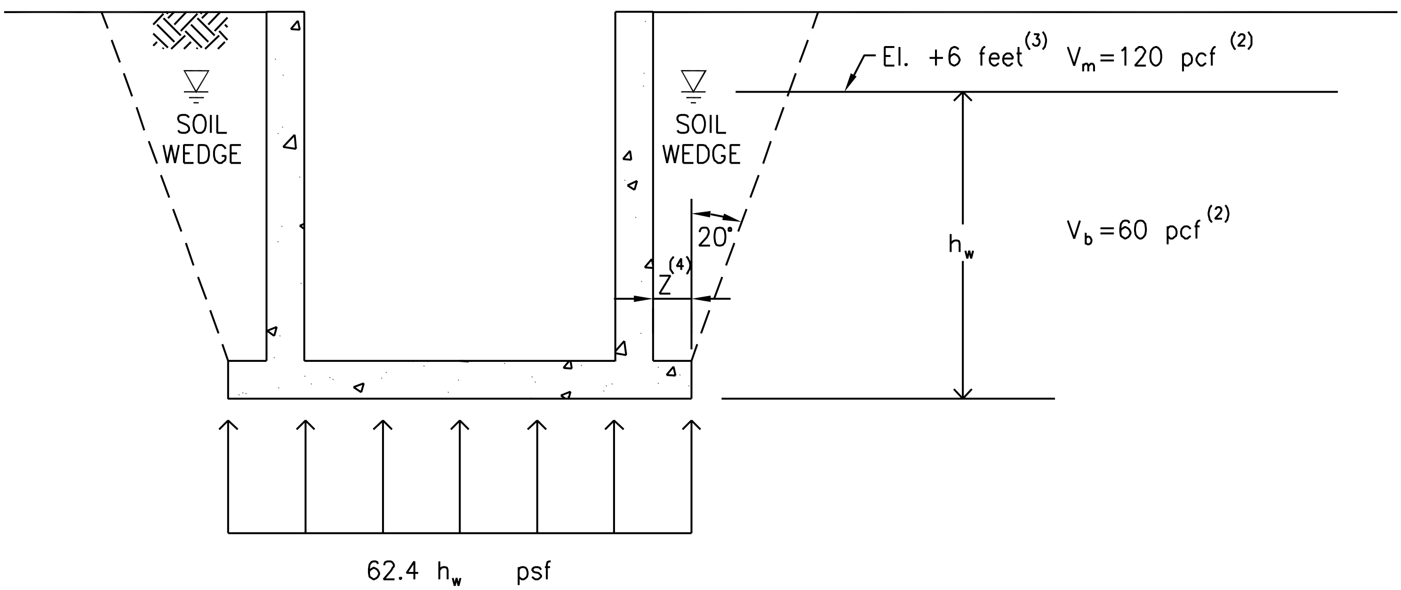
For wall penetrations at pipe locations, seals that limit the amount of seepage to an acceptable level should be designed and installed. Water stops should be used at horizontal and vertical construction joints to reduce the likelihood of water infiltration. Waterproofing should be protected from being damaged by compaction equipment and other construction vehicles after installation, and any damage should be repaired prior to resuming with the backfilling operations.

9.0 PAVEMENT DESIGN

9.1 **Flexible Pavement.** To facilitate vehicle access and parking, the proposed power plant development will include paving in the areas surrounding facility structures. Near surface on-site soils exhibit a high traffic supporting strength when used as pavement subgrade. R-values of 45, 80, and 85 were measured from representative bulk samples of near-surface site soil. An R-Value of 45 was utilized for determining the asphalt concrete (AC), aggregate base (AB), and aggregate subbase (ASB) sections presented on Table 7 – AC Pavement Structural Sections. The structural sections presented in the table are based on the California Method of flexible pavement design as presented in Chapter 600 of the Caltrans Highway Design Manual (Caltrans, 2004). The pavement sections consider traffic indices of 5, 7, and 9 for the possible type and volume of traffic anticipated on the site access/maintenance roads and parking areas.

FIGURE 8

UPLIFT RESISTANCE[1]



HYDROSTATIC
UPLIFT PRESSURE

NOTES:

(1) UPLIFT RESISTANCE = WEIGHT OF STRUCTURE + WEIGHT OF SOIL WEDGES.

(2) V_m AND V_b ARE UNIT WEIGHTS OF BACKFILL SOILS ABOVE THE GROUNDWATER TABLE AND BELOW THE GROUNDWATER TABLE, RESPECTIVELY.

(3) ASSUMES WALLS AND SLAB ARE NOT DRAINED SO THAT HYDROSTATIC PRESSURES BUILD UP TO THE DESIGN GROUNDWATER ELEVATION OF +6 FEET (NAVD 1988).

(4) WIDTH OF FOOTING KEY, Z, SHOULD BE AT LEAST 12 INCHES TO MOBILIZE SOIL WEDGES.

TABLE 7 –AC PAVEMENT STRUCTURAL SECTIONS

Traffic Index	5		7		9	
	Option 1	Option 2	Option 1	Option 2	Option 1	Option 2
Asphalt Concrete	3”	2.5”	4”	3.5”	6”	5”
Class 2 Aggregate Base (R=78)	4”	5”	6”	6”	8”	8”
Aggregate Sub-base (R=50)	-	4”	-	4”	-	4”

9.2 **Rigid Pavement.** Rigid pavement, or Portland cement concrete (PCC) pavement, should be designed in accordance with the Caltrans Highway Design Manual (2004). Topic 603 of the referenced manual covers the design of the PCC pavement structural section, and Topic 606 covers the design of the drainage components.

The Resistance Value, or R-value, of the basement soils was tested by California Test 301. Three soil samples in the upper 5 feet of fill were evaluated to have R-values of 80, 85 and 45.

Based on the results of the R-value testing and the Caltrans Highway Design Manual, we recommend the PCC pavement structural sections provided in Table 8 – PCC Pavement Structural Sections.

TABLE 8 – PCC PAVEMENT STRUCTURAL SECTIONS

Traffic Index	PCC Pavement Thickness (inches)	Treated Permeable Base Thickness (inches)	Aggregate Base Thickness (inches)
5	8	4	4
7	8	4	4
9	8.5	4	5

Structural section components should comply with the relevant portions of the Caltrans Standard Specifications (1999) along with the Amendments to the July 1999 Standard Specifications (2005) except as modified in this section of the report. The PCC pavement should have a compressive strength of at least 4,000 psi, and have at least 5.5 sacks of cement per cubic yard of concrete. Treated permeable base may be either asphalt treated (ATPB) or cement treated (CTPB) in accordance with Section 29 of the Standard Specifications. Aggregate base material should meet the requirements of Class 2 Aggregate Base in Section 26 of the Standard Specifications. The aggregate base should be compacted to at least 95 percent of the maximum dry density as evaluated by ASTM D1557.

The treated permeable base should have a free draining outlet either to a daylight or to an underground piping system so that the basement soils do not get saturated.

If differential movement and/or some minor spalling at construction joints is undesirable, dowel bars should be installed to connect adjacent sections of PCC pavement. The dowels should be a smooth, epoxy-coated 1 ¼ inch bar, and set across the pavement joints so that the dowel moves independently from the PCC pavement. The spacing of the dowels should be about every one foot along the joint. Additional guidance can be provided in the event that dowels are deemed to be necessary at joints.

10.0 SITE VIBRATION STUDY

We understand that the vibrations caused by the fuel gas compressors at the project site are of concern to the designers of the neighboring Muni Metro East facility. We carried out a vibration study of the four fuel gas compressors at the southwestern portion of the site.

PB Power, Inc. provided fuel gas compressor data for an Ariel JGD/2 compressor. The Ariel JGD/2 compressor is a large, medium speed, reciprocating, horizontal compressor with two double-acting cylinders. The compressors operate at a maximum speed of 1200 revolutions per minute. The weight of the compressor skid was estimated to be 100 kips by PB Power, Inc., and the skid dimensions are approximately 14 feet wide by 32 feet long by 10 feet high.

The two cylinders in each of the compressors are of two types: cylinder model 15-7/8D, the low pressure cylinder, and cylinder model 11D, the high pressure cylinder. The cylinder models refer to the bore diameter in inches. The cylinders are opposed and the weights balanced so that the unbalanced forces of the reciprocating compressors are largely cancelled out. However, the fuel gas compressors can operate with some imbalance on the opposing cylinders in between maintenance cycles. Small unbalanced horizontal forces are created by this imbalance, which cause the equipment to be pushed and pulled in translation. Though unbalanced forces are small to negligible, two cylinder, reciprocating compressors can have significant horizontal primary and secondary couples and significant vertical primary couples. The horizontal couples are created by the need to horizontally offset the opposing cylinders, which results in twisting of the equipment. This is sometimes referred to as “yawing”. The vertical couple is created by the vertical component of motion of the counterweight on the crankshaft arm. This results in rocking of the equipment. Table 9- Unbalanced Forces and Couples Data for Ariel JGD/2 Compressor provides the unbalanced forces and couples data for the Ariel JGD/2 compressor as provided by PB Power, Inc.

TABLE 9 – UNBALANCED FORCES & COUPLES DATA FOR ARIEL JGD/2 COMPRESSOR

Direction of Force	Description	Primary	Secondary
Horizontal	Force at Maximum Imbalanced Weight of 2.5 lbs.	0.274 kip	0.044 kip
	Force for Actual Weights	0.066 kip	0.011 kip
	Couple	20.437 kip ft.	6.867 kip ft.
Vertical	Force	0	--
	Couple	22.016 kip ft.	--

The unbalanced forces and couples data was used to analyze the likely vibration response of the fuel gas compressor foundations. We analyzed the dynamic response of the foundations using steady-state, frequency-dependent input motions from the reciprocating compressors. The compressor and foundation system were lumped together as a single mass with a single spring and single damping constant for each mode of vibration (translation, rocking and twisting), and the ground was assumed to be an elastic half space. The simplifications of this lumped-parameter system approach allow for easy use of parametric studies to select an appropriate foundation design. The approach, with relatively minor variations between references, is included in Richart et al. (1970), Bowles (1996), and Naval Facilities Engineering Command (1983).

The soil was modeled based on soil classification from borings B-10 and B-11, laboratory test results, and the cross-hole seismic test results performed in the vicinity of the proposed fuel gas compressors. Supporting geotechnical data is provided in Appendix A. The geophysics report by Southwest Geophysics, Inc., with the crosshole seismic data, is included in Appendix C. The upper 15 to 30 feet of the soil profile, along with deep foundation systems when used, affects the dynamic response of the compressor foundations. The upper 10 feet of soil is most significant. The crosshole seismic tests indicate that the shear wave velocity, V_s, of the fill is high in the upper 10 feet ($V_{s,\ avg} \approx$ 1280 ft./sec.), increases for the next 10 feet ($V_{s,\ avg} \approx$ 1590 ft./sec.), and then drops to lower values in the bottom 10 feet of fill ($V_{s,\ avg} \approx$ 980 ft./sec.). The soil parameters used in the dynamic response of foundations is shown in Table 10 – Soil Parameters for Dynamic Response of Foundations.

TABLE 10 – SOIL PARAMETERS FOR DYNAMIC RESPONSE OF FOUNDATIONS

Soil Parameter	Value
Unit weight, γ	130 pcf
Shear wave velocity, $V_{s,\,max}$	1,280 ft./sec.
Shear modulus, G_{max}	6620 ksf
Poisson's ratio, ν	0.33
Damping ratio, D	0.03

Generally, the design of machine foundations is an iterative process wherein a foundation type is first assumed, and the vibrations calculated. The foundation is then modified until the vibrations are in an acceptable range and the natural frequency of the equipment/foundation system is far from the operating frequencies of the machine. The maximum operating frequency of the Ariel JGD/2 fuel gas compressors is 1200 revolutions per minute, or 20 hertz. The compressors can therefore be expected to operate at between about 10 to 20 hertz with lower frequencies passed through during equipment start up and shut down. Due to the high shear wave velocities of the fill and the large plan area of the compressor skid, the natural frequency of the lumped-mass system is fairly high. For this reason, foundation modifications on subsequent iterations were to add extra stiffness and further increase the natural frequencies.

Based on our dynamic response analysis, we have the following geotechnical recommendations for the fuel gas compressor foundations:

- The foundations for the compressor skids are designed to be pile caps supported by 14-inch square friction piles based on supporting static loads and accounting for possible site settlement. We recommend that the dead load on the piles for the fuel gas compressors be limited to one-half of the allowable geotechnical capacities provided in the Foundation Recommendations section. The center to center spacing of the piles should be at least 6 feet. The piles should be well anchored to the pile cap so that pile fixity at the top is ensured. Further, the pile layout should be carefully selected to arrange the piles in a pattern about the center of gravity of the compressor skid plus pile cap.
- The pile cap should be 30 inches thick for vibration considerations. If the pile cap is required to be thinner or thicker for other considerations, further vibration analysis should be performed with the revised pile cap thickness.
- The vibration response is based on the subgrade below the pile cap consisting of well-compacted gravelly soils with a minor amount of fines. If during grading, on-site soil conditions are judged to be different from this, the upper 24 inches of soil should be removed and replaced with Class 2 aggregate base (Caltrans Standard Specifications, 1999). The Class 2 aggregate base and/or upper 24 inches of gravelly soils should be compacted to 95 percent of maximum dry

density as evaluated by ASTM D1557. Modification of the on-site soils with cement or lime additives in lieu of replacement may also be considered by the geotechnical engineer during construction. Furthermore, the site grading should be done in such a way that there is no settlement of the soil away from the pile-supported pile cap because of consolidation-related settlement in the underlying younger bay mud.

Provided the above foundation recommendations are followed, we estimate that the amplitude of vibration at the foundation level from a single gas compressor will be no more than approximately 0.0001 inch. The predominant frequencies of vibration are anticipated to be between about 20 and 40 hertz based on an operating speed of 10 to 20 hertz of a double-acting cylinder. Since there may be up to four fuel gas compressors operating at any one time, there may be some superposition of vibrations transmitted away from the fuel gas compressor foundations. The largest amplitude of motion by superposition for the proposed system is estimated to be on the order of 0.0004 inch, though it will likely be less as the vibrations will be at different phases from each of the four compressors. Based on data compiled by Reiher and Meister (1931) on the human perception of steady state vibrations, the vibrations in the range of 0.0001 inch to 0.0004 inch will be barely to easily noticeable to persons if standing adjacent to the fuel gas compressors.

For off-site structures at the adjacent Muni Metro East facility, the vibration levels will be less than those immediately adjacent to the compressors. The vibration level is less due to both geometrical damping as it propagates radially away from the source, and by material damping within the soil. The vibrations decay proportionally to the square root of the distance away from the source due to geometrical damping, and a lesser amount due to internal damping in soil. Our calculations indicate that the displacement amplitudes at a distance of 50 feet from the source will be on the order of ¼ to ½ of the displacement amplitudes at the source of vibration. This indicates that the amplitude of vibration at the Muni facility will be no more than approximately 0.0001 to 0.0002 inch provided the foundation recommendations are followed. Vibrations of this magnitude at a frequency of about 20 to 40 hertz are barely noticeable to persons.

The allowable vibration levels at the Muni Metro East facility were unknown at the time of submission of this report. If vibration levels less than those estimated herein are required, additional dynamic response analysis of foundations can be performed.

11.0 GEOPHYSICS STUDY

We contracted with Southwest Geophysics, Inc. to perform geophysical testing at the site. The geophysical testing was performed for two main purposes: (1) to evaluate the presence and distribution of subsurface obstructions in the fill across the four-acre site and (2) to evaluate the dynamic properties of the soils for a machine vibration study and for a site-specific dynamic response analysis during earthquakes. The geophysics report includes a description of the geophysical methods utilized, the test results and an analysis of the test results (Appendix C).

Southwest Geophysics, Inc. concludes that there is evidence of buried subsurface obstructions in the upper 5 feet of fill, as well as buried deeper within the fill. The high resistivity areas depicted in Figure 5 of the geophysics report (brown, orange, red and purple shades) may indicate greater quantities of concrete and brick rubble and similar highly-resistive materials. The data is representative for the resistivity lines STL-1 and STL-2 as shown on Figure 2 of the geophysics report and on Plate 2 – Field Exploration Map. The results, though, show that subsurface debris is prevalent over large portions of the site.

Crosshole and downhole geophysical testing was performed to evaluate the shear wave velocity of the soil profile. The results are shown on Figure 7 of the geophysics report. This data was utilized in the machine vibration study and the site-specific dynamic response analysis during earthquakes. A refraction microtremor (ReMi) survey was also conducted and results presented in Figure 6 of the geophysics report. The ReMi test results show a reasonable average trend of shear wave velocities with depth, but were considered to be not as reliable as the crosshole and downhole geophysical test results, and were therefore given lesser weight in selection of dynamic soil parameters.

12.0 CORROSION

Marine environments such as that of the project site are typically moderately to highly corrosive to ferrous materials because of the presence of salt water and microbes in the bay mud. Microbial corrosion tends to be most serious in poorly drained soils that have low oxygen levels and redox potentials, high organic matter levels, high clay contents, and neutral pH values (Iverson, 1974). Hence the marine deposits underlying the artificial fill at the site should be considered moderately to severely corrosive to structural elements composed of ferrous materials. Elements extending above the ground surface will be exposed to a salt water environment and should be protected from corrosion as appropriate.

To further characterize the corrosive properties of the areal fill, the bay mud, and the upper layered sediments at the project site, representative soil samples were collected from our borings B-6, B-8, B-11, and B-12 at depths of 14, 45, 5, and 65 feet, respectively, and tested for corrosive properties. Each sample was tested for sulfates, chlorides, pH, and resistivity in as-received and saturated states. The tests were carried out by CONCECO/MATCOR Engineering, Inc. Results of the laboratory tests for corrosive properties are included in Appendix A.

The resistivity tests indicate that the younger bay mud sample (recovered from B-8) and the upper layered sediments sample (recovered from B-12) are severely corrosive to ferrous materials and the fill samples are slightly corrosive to moderately corrosive. The sulfate and chloride content in the sample recovered from B-8 at 45-foot depth are very high, indicating that the soils are not only severely corrosive to ferrous materials but also deleterious to concrete. Chlorides are particularly corrosive to ferrous materials. The pre-stressed concrete piles' mix design should include appropriate corrosion inhibitors against sulfate and chloride attack to maintain the design life of the piles. The detailing of the lifting lugs in the pile should adequately address the prevention of water from getting into the interior of the pile, such as the use of a suitable epoxy to seal the lug locations.

13.0 ENVIRONMENTAL ANALYSIS

During our subsurface investigation, CH2M Hill collected soil samples for an environmental conditions analysis. The samples were collected from select borings. It is our understanding that CH2M Hill will provide the laboratory results to the SFPUC under separate cover.

14.0 CONSTRUCTION CONSIDERATIONS

Excavations will most likely encounter some zones containing large debris such as concrete blocks, wood, and metal requiring additional means for removal, though measures such as heavy ripping or blasting are not anticipated. Excavations that are greater than 6 feet deep may encounter groundwater that may inundate the excavation, requiring dewatering measures. Dewatering measures should be implemented to provide a relatively dry environment for the placement, moisture conditioning, and compaction of engineered fill and backfill, and to provide a firm working surface at foundation grades for construction of footings or other soil load bearing structures. Design and implementation of any dewatering scheme should be the responsibility of the contractor.

Excavation-related settlement should be evaluated by the contractor when performed in close proximity to adjacent structures. The settlements related to excavation support movements, bottom heave, and dewatering should be evaluated by a qualified geotechnical engineer. Pre-condition surveys of existing structures may also be performed to document existing distress to the structures.

Installation of pile foundations is likely to be difficult due to the prevalence of buried debris within the fill layer. Pre-drilling for piles will be required, and may require considerable effort on the part of the pile driving contractor. The pre-drilling will encounter obstructions that will be difficult to bypass, and caving will likely occur during pre-drilling below the groundwater table.

Pile driving criteria should be developed using an appropriate wave equation type dynamic analysis based on the contractor's proposed driving equipment and the pile type, prior to pile installation. The upper layered sediments through which the piles must be driven have layers of dense to very dense sands, so selection of an appropriate hammer for pile installation is essential.

Pile driving will also cause considerable noise and vibrations, which can sometimes be disturbing to people working in the area. Typically the vibrations do not cause distress to structures as long as the pile driving takes place more than about 20 to 30 feet from it. As a precaution, we recommend that piles first be installed far from adjacent buildings while recording ground vibrations with a seismograph. The need for continuing use of the seismograph during pile driving can be evaluated by the geotechnical engineer during construction.

We should be retained during construction to provide site observation and consultation concerning the condition of the bottom of excavations pertaining to foundation construction, and for pile driving observation and documentation. Foundation grades should be observed and, where necessary, tested under the direction of a qualified geotechnical engineer to verify compliance with final design recommendations. All site preparation work and excavations should also be observed to compare the generalized site conditions assumed in the final design report with those found on site at the time of construction.

15.0 CLOSURE

The conclusions and recommendations presented herein are professional opinions based on geotechnical and geologic data and the project as described. A review by this office of any foundation, excavation, grading plans and specifications, or other work product that relies on the content of this report, together with the opportunity to make supplemental recommendations is considered an integral part of this study. Should unanticipated conditions come to light during project development or should the project change from that described, we should be given the opportunity to review our recommendations.

The findings and professional opinions presented in this report are presented within the limits prescribed by the client, in accordance with generally accepted professional engineering and geologic practices. There is no other warranty, either express or implied.

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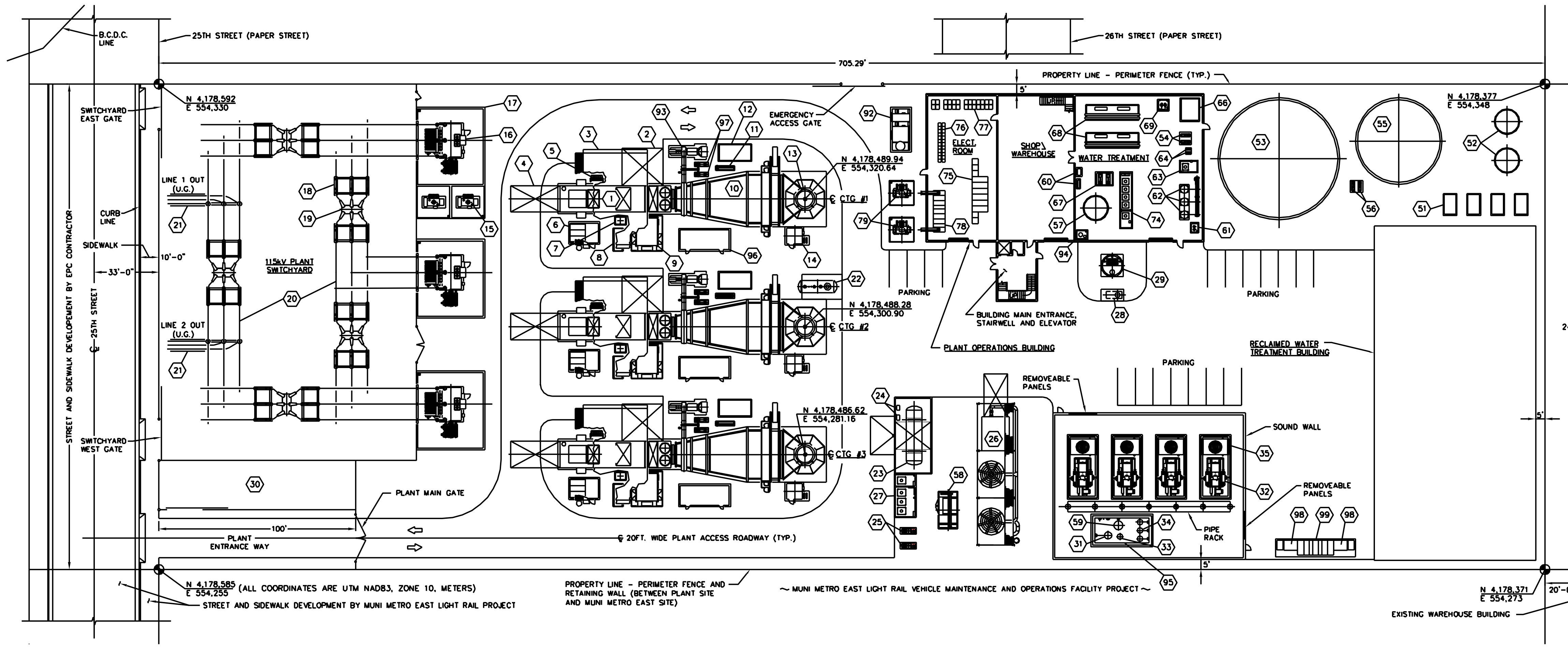

PLOT PLAN

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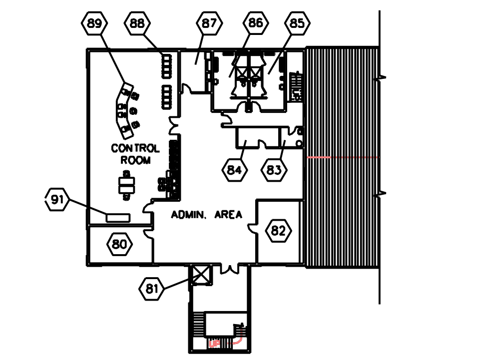

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PLANT OPERATIONS BUILDING
2ND FLOOR – PLAN

LEGEND:

TYPICAL FOR THREE UNITS (items 1–14)

1. LM6000 COMBUSTION TURBINE GENERATOR
2. TURBINE REMOVAL/MAINTENANCE AREA
3. CTG AIR INTAKE FILTER SYSTEM
4. GENERATOR ROTOR REMOVAL AREA
5. CTG FIRE PROTECTION SKID
6. GENERATOR BREAKER SWITCHGEAR
7. SPRINT SYSTEM SKID
8. NOx WATER INJECTION SKID
9. AUXILIARY SKID
10. SCR/CO CATALYST SYSTEM
11. AMMONIA FLOW BALANCE SKID
12. AMMONIA VAPORIZATION SKID
13. STACK
14. CEMS
15. 5KV AUXILIARY TRANSFORMER (TYP. 2)
16. 13.8kV/115KV GSU (TYP. 3)
17. FIRE/BLAST WALL (TYP.)
18. 115kV SWITCH (TYP. 10)
19. 115 kV BREAKER (TYP.5)
20. SWITCHYARD BUS WORK
21. 115kV DUCT BANK
22. TURBINE WASH WATER DRAIN TANK (UG)
23. AQUEOUS AMMONIA STORAGE TANK
24. AQUEOUS AMMONIA FORWARDING PUMPS
25. AUXILIARY COOLING PUMPS
26. CHILLER/COOLING TOWER PACKAGE
27. COOLING TOWER CHEMICAL SYSTEM
28. OIL/WATER SEPARATOR (UG)
29. WASTE WATER SUMP AND LIFT STATION
30. 100'x25' PG&E GAS METERING STATION
31. NATURAL GAS INLET SCRUBBER
32. FUEL GAS COMPRESSOR (TYP. 4)
33. HYDROCARBON DRAIN TANK
34. DISCHARGE FILTER SCRUBBER (TYP. 2)
35. FUEL GAS COOLING RADIATOR (TYP.4)
36.–50. (HOLD)
51. ODOR CONTROL FANS (TYP. 4)
52. ODOR CONTROL SCRUBBERS
53. TREATED WATER STORAGE TANK
54. TREATED WATER PUMPS
55. DI WATER STORAGE TANK
56. DI WATER PUMPS
57. RO BREAK TANK
58. ANTI-ICING HEATER PACKAGE
59. ACCUMULATOR
60. DI SYSTEM CONTROL PANELS
61. RESIN FILTERS
62. LEASED DEMINERALIZER VESSELS
63. SODIUM HYPOCHLORITE METERING SYSTEM
64. DOMESTIC NON-POTABLE WATER PUMPS
65. (NOT USED)
66. RO CLEAN IN PLACE SKID
67. 2nd PASS RO FEED PUMP SKID
68. RO TRAINS
69. RO CARTRIDGE FILTER SKID
70. (NOT USED)
71. (NOT USED)
72. (NOT USED)
73. (NOT USED)
74. DI WATER CHEMICAL METERING SYSTEM
75. 5kV SWITCHGEAR
76. 480V MCC'S
77. BATTERIES
78. 480V SWITCHGEAR
79. 480V STATION SERVICE TRANSFORMERS
80. PRIVATE OFFICE
81. ELEVATOR
82. CONFERENCE/TRAINING ROOM
83. JANITOR'S STORAGE
84. OFFICE SUPPLY STORAGE
85. MEN'S LOCKERS/SHOWER
86. WOMEN'S LOCKERS/SHOWER
87. LUNCH ROOM
88. INPUT/OUTPUT CABINETS
89. HUMAN/MACHINE INTERFACE
90. CTG CONTROL PANELS
91. SWITCHYARD CONTROL PANEL
92. AIR COMPRESSOR SKID
93. PURGE AIR FAN (TYP. AT EACH SCR)
94. TEMPERED WATER SKID
95. FUEL GAS CONDITIONING SKID
96. POWER AND CONTROL CAB (PCC) (TYP. EA. CTG)
97. DAMPER SEAL AIR FANS (TYP. AT EACH SCR)
98. 480V DRY TYPE TRANSFORMER
99. 480V SWITCHGEAR, RECLAIMED WATER AREA

E	8-30-05	WATER TREATMENT AND RECLAIMED WATER AREAS REVISED
D	8-9-05	ITEMS 96 THRU 99 ADDED AND ITEM 22 RELOCATED
C	8-3-05	EMERG. GATE & SECT. E ADDED
B	6-30-05	GAS METERING STATION RELOCATED
A	6-14-05	PRELIMINARY ISSUE

REFERENCE:
"Muni Site Plot Plan" Drawing No. G1.2, San Francisco Electric Reliability Project, by PB Power, Inc.

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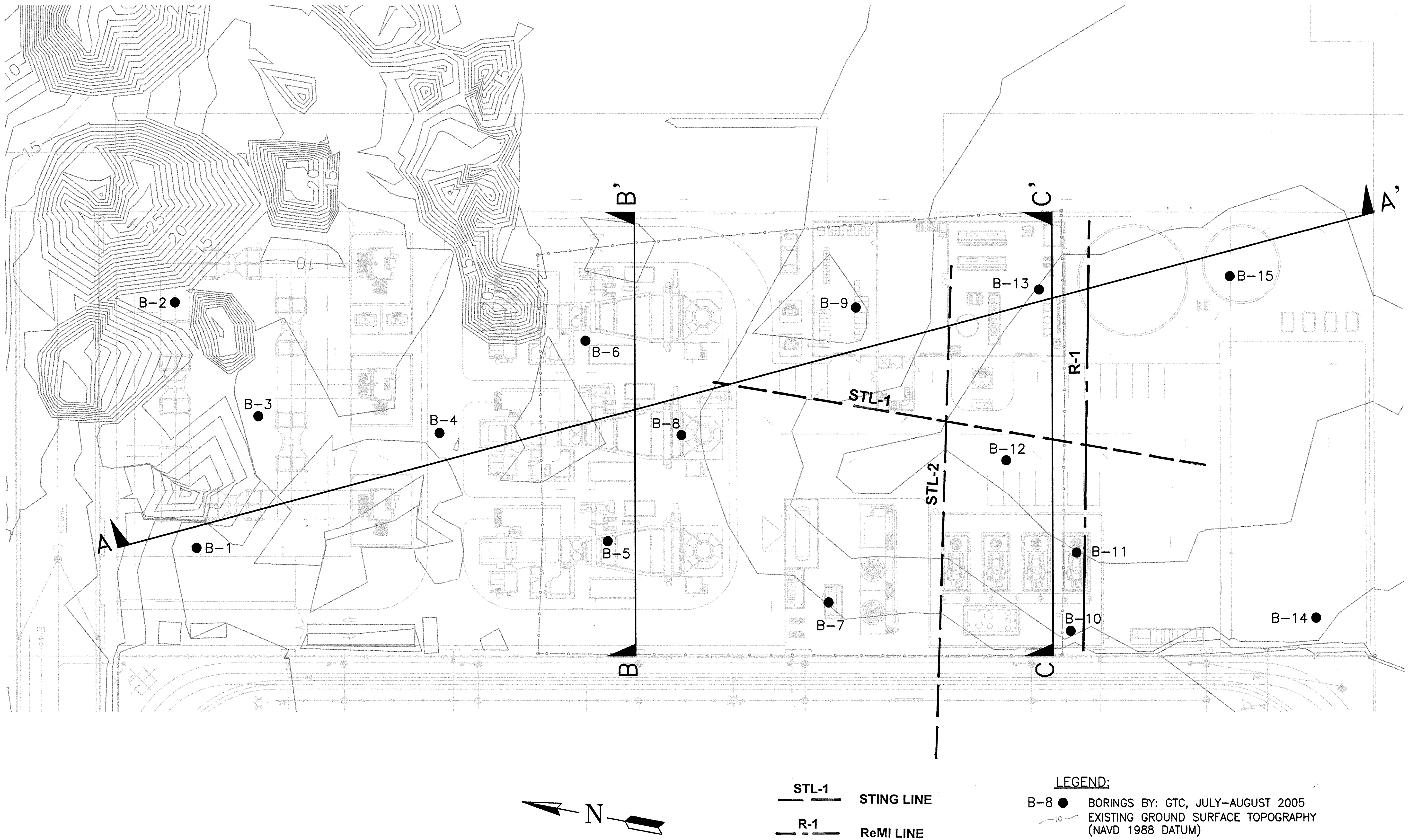
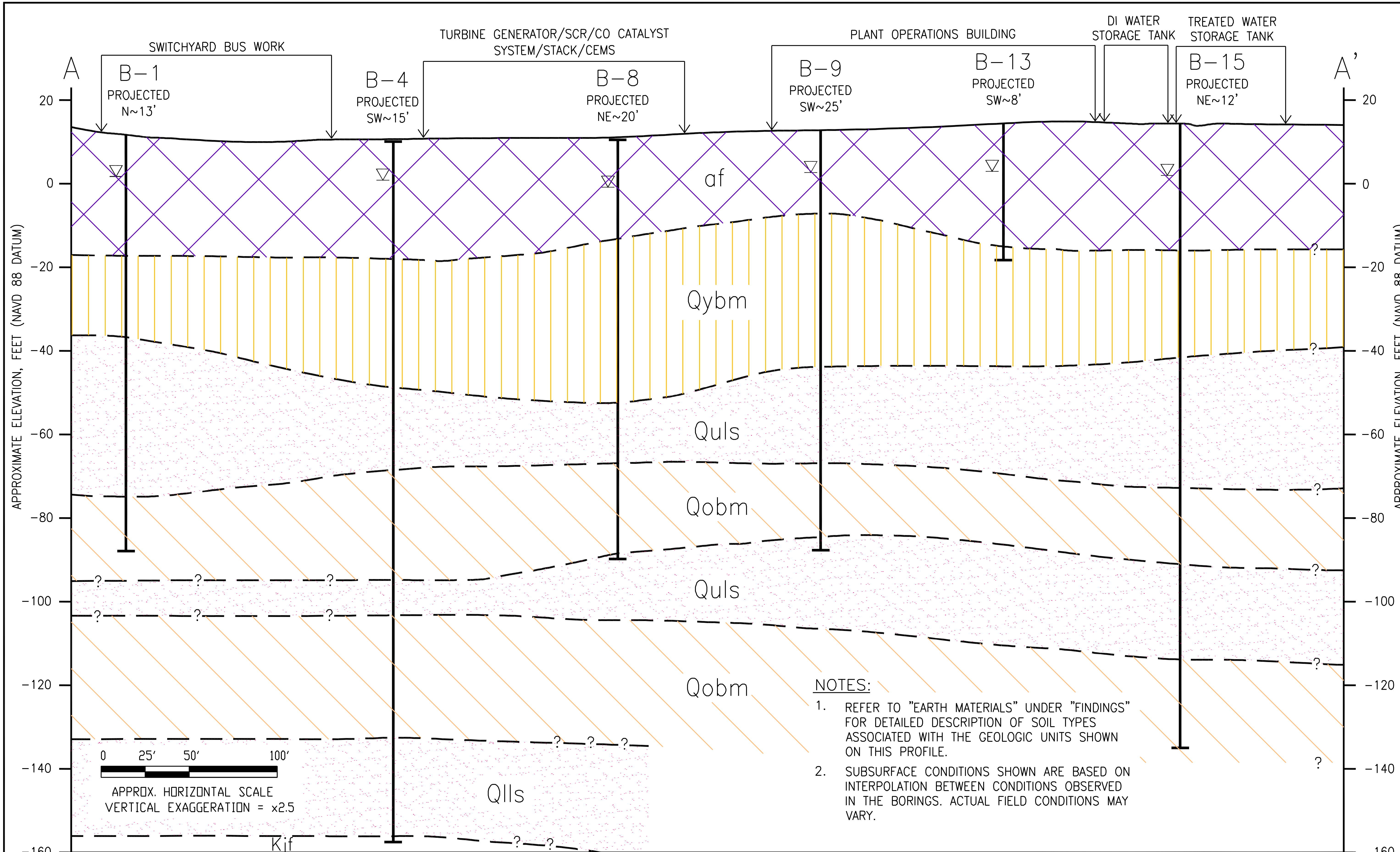
San Francisco, CA 94111

Site Plan	PLATE1
Muni Site	Oct. 2005
SFPUC ERP Power Plant	SF05019



NOTE:
Base Map provided by BP POWER, Inc.
Reference: "MUNI SITE-PLOT PLAN-3 UNITS SIMPLE CYCLE", SFERP, San Francisco, California
DRAWING NO. C1, Preliminary Issue

GEOTECHNICAL CONSULTANTS, INC. 500 Sansome Street, Suite 402 San Francisco, CA 94111	Field Exploration Map	PLATE 2
	Muni Site	Oct. 2005
	SFPUC ERP Power Plant	SF05019



NOTES:

1. REFER TO "EARTH MATERIALS" UNDER "FINDINGS" FOR DETAILED DESCRIPTION OF SOIL TYPES ASSOCIATED WITH THE GEOLOGIC UNITS SHOWN ON THIS PROFILE.
2. SUBSURFACE CONDITIONS SHOWN ARE BASED ON INTERPOLATION BETWEEN CONDITIONS OBSERVED IN THE BORINGS. ACTUAL FIELD CONDITIONS MAY VARY.

KEY:

- ∇ GROUND WATER
- Qlls LOWER LAYERED SEDIMENTS
- Kjf FRANCISCAN COMPLEX BEDROCK
- af ARTIFICIAL FILL
- Qybm YOUNGER BAY MUD
- Quls UPPER LAYERED SEDIMENTS
- Qobm OLDER BAY MUD

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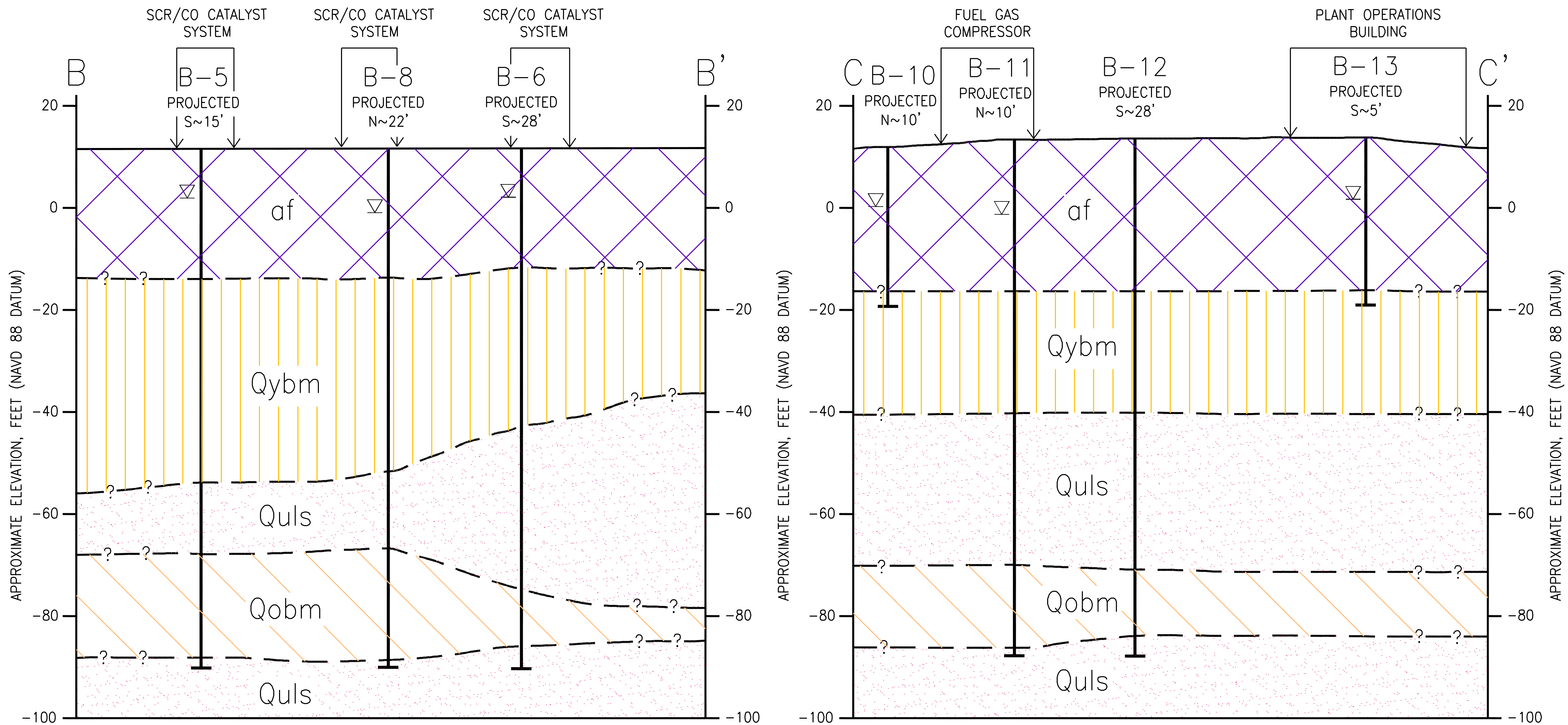
SUBSURFACE PROFILE A-A'	PLATE 3
Muni Site	Oct. 2005
SFPUC ERP Power Plant	SF05019



0 20' 40' 80'

APPROX. HORIZONTAL SCALE
VERTICAL EXAGGERATION = x2

NOTES:

1. REFER TO "EARTH MATERIALS" UNDER "FINDINGS" FOR DETAILED DESCRIPTION OF SOIL TYPES ASSOCIATED WITH THE GEOLOGIC UNITS SHOWN ON THIS PROFILE.
2. SUBSURFACE CONDITIONS SHOWN ARE BASED ON INTERPOLATION BETWEEN CONDITIONS OBSERVED IN THE BORINGS. ACTUAL FIELD CONDITIONS MAY VARY.

KEY:

- ▽ GROUNDWATER
- Kjf FRANCISCAN COMPLEX BEDROCK
- af ARTIFICIAL FILL
- Qybm YOUNGER BAY MUD
- Quls UPPER LAYERED SEDIMENTS
- Qobm OLDER BAY MUD

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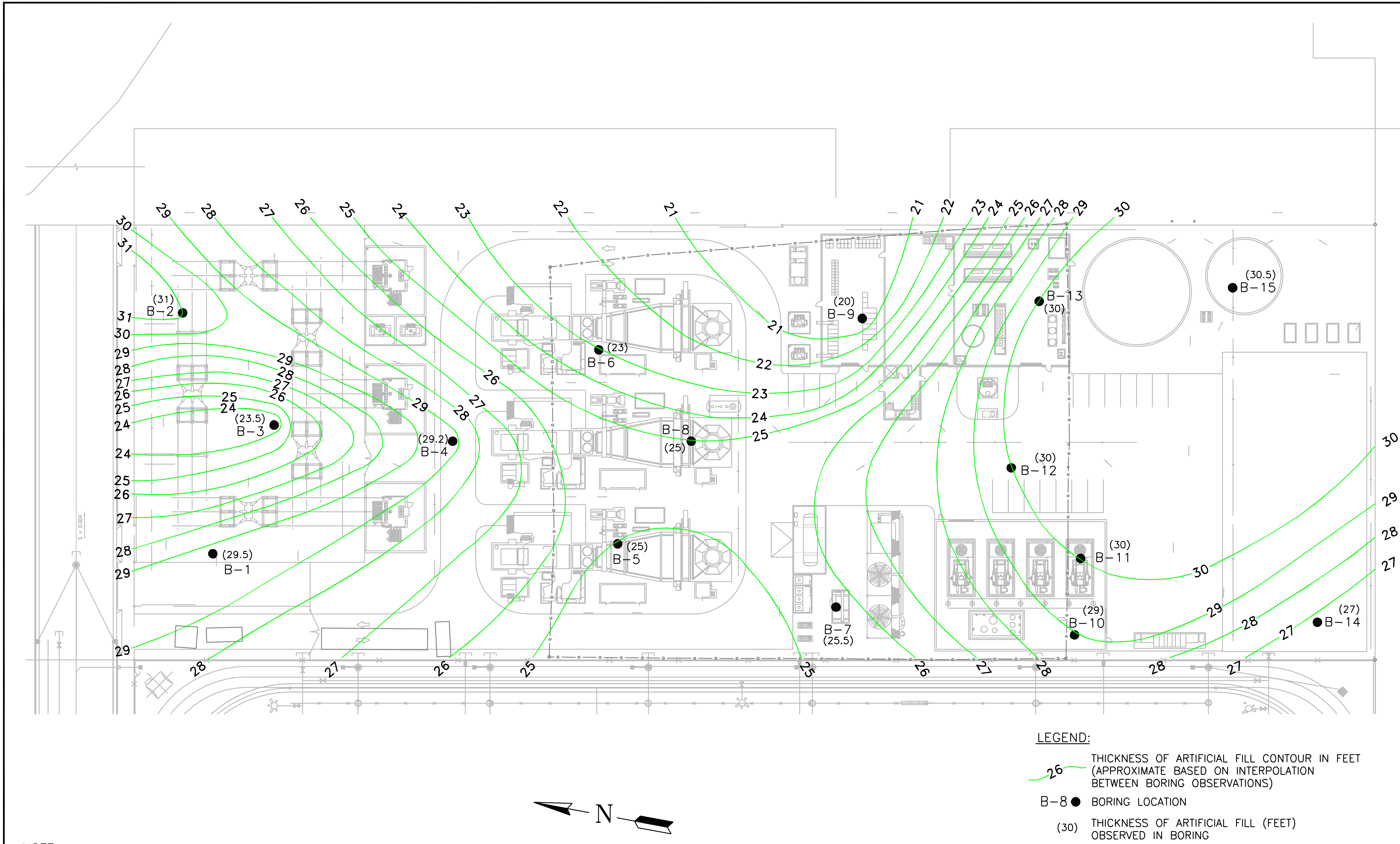
SUBSURFACE PROFILE B-B', C-C'	PLATE 4
Muni Site	Oct. 2005
SFPUC ERP Power Plant	SF05019



NOTE:
Base Map provided by BP POWER, Inc.
Reference: "MUNI SITE-PLOT PLAN-3 UNITS SIMPLE CYCLE",
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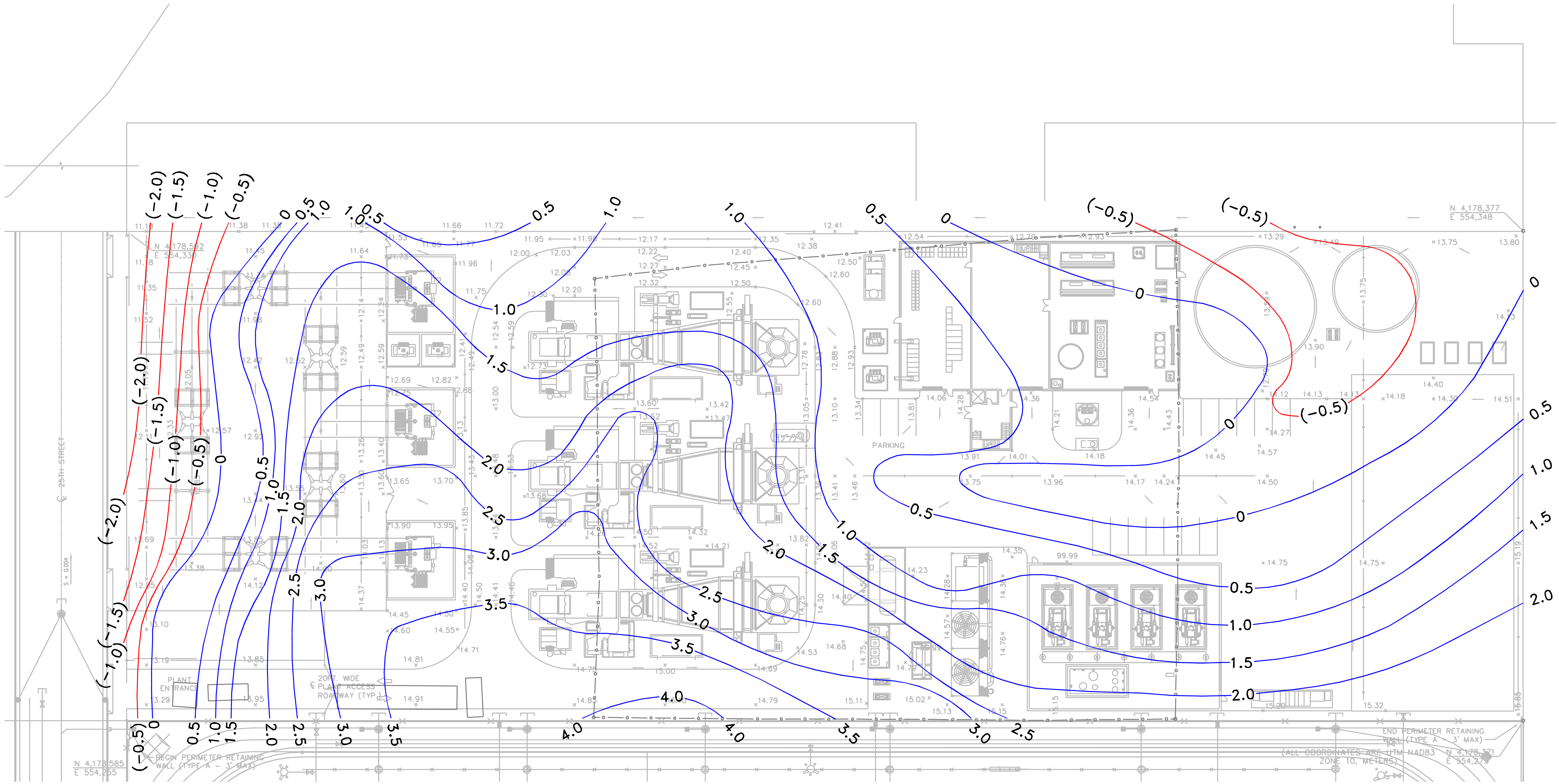
Thickness of Existing Artificial Fill	PLATE 5
Muni Site	Oct. 2005
SFPUC ERP Power Plant	SF05019



LEGEND:

×14.75 PROPOSED DESIGN GRADE SPOT ELEVATION (FEET, NAVD 1988 DATUM)

1.5 THICKNESS OF NEW FILL CONTOUR, IN FEET (APPROXIMATE BASED ON SITE SURVEY TOPOGRAPHY AND PROPOSED DESIGN GRADE SPOT ELEVATIONS PROVIDED BY PB POWER, INC.

(−0.5) THICKNESS OF CUT CONTOUR, IN FEET (APPROXIMATE BASED ON SITE SURVEY TOPOGRAPHY AND DESIGN GRADE SPOT ELEVATIONS PROVIDED BY PB POWER, INC.

N

NOTE:
Base Map provided by BP POWER, Inc.
Reference: "MUNI SITE-PLOT PLAN-3 UNITS SIMPLE CYCLE", SFERP, San Francisco, California
DRAWING NO. C1, Preliminary Issue

60' 30' 0 60' 120'

SCALE: 1" = 60'

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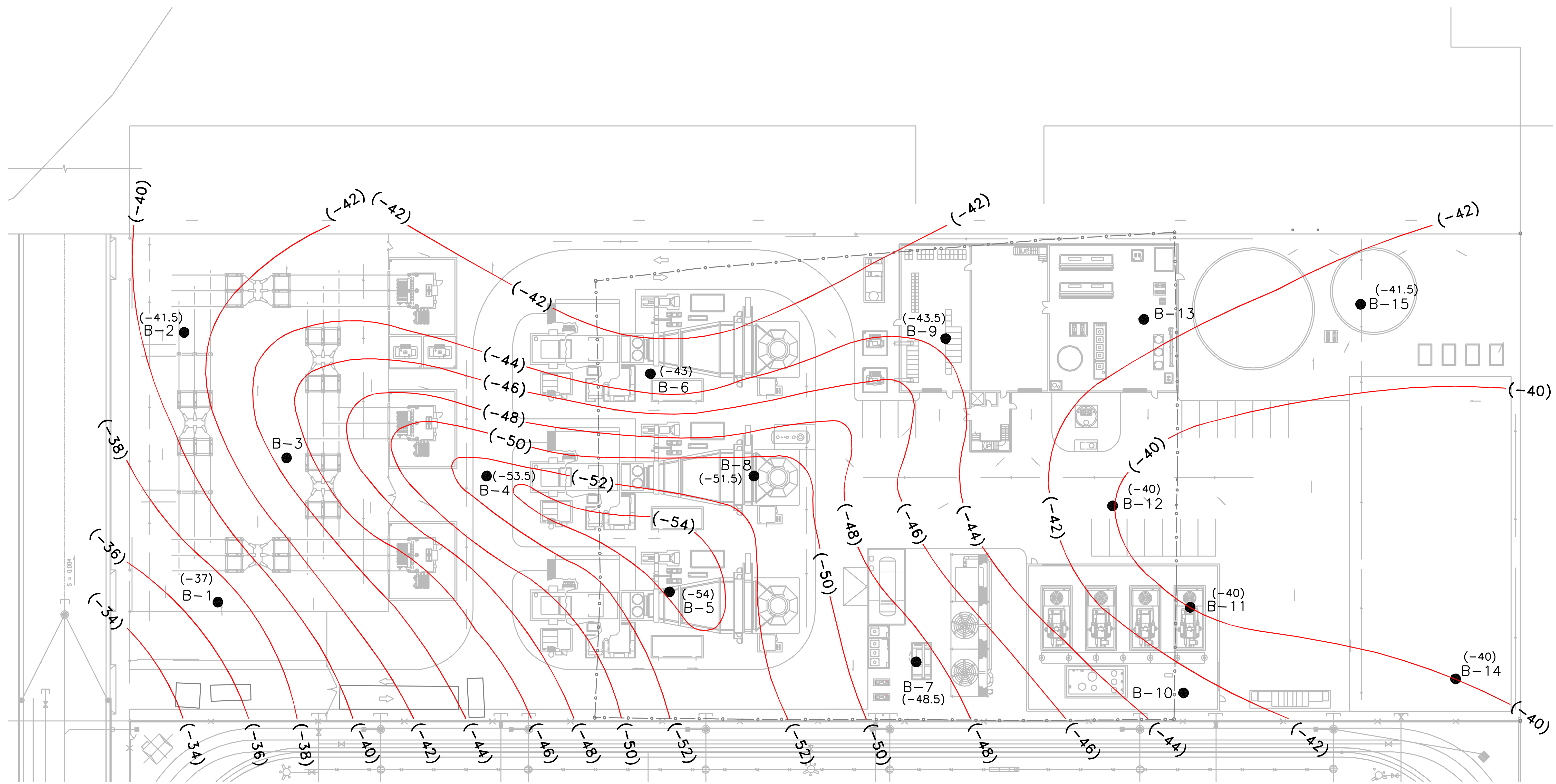
Thickness Contours of Proposed Cut and Fill	PLATE 6
Muni Site	Oct. 2005
SFPUC ERP Power Plant	SF05019



LEGEND:

(-48) BOTTOM OF YOUNGER BAY MUD ELEVATION CONTOUR, FEET, NAVD 1988 DATUM (APPROXIMATE, BASED ON INTERPOLATION BETWEEN BORING OBSERVATIONS)

B-8 ● BORING LOCATION

(-40) BOTTOM OF YOUNGER BAY MUD ELEVATION OBSERVED

NOTE:
Base Map provided by BP POWER, Inc.
Reference: "MUNI SITE-PLOT PLAN-3 UNITS SIMPLE CYCLE",
SFERP, San Francisco, California
DRAWING NO. C1, Preliminary Issue

60' 30' 0 60' 120'

SCALE: 1" = 60'

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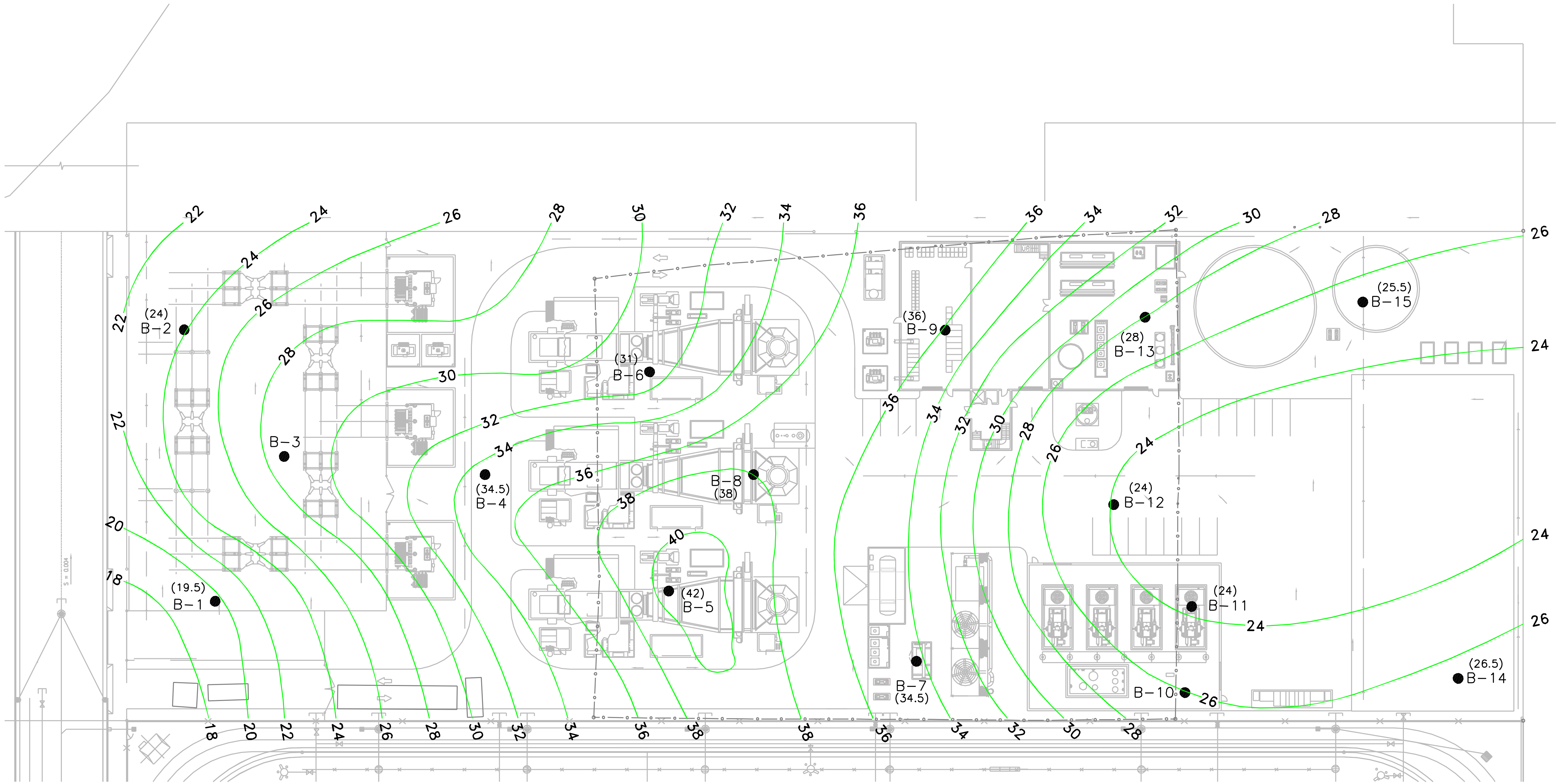
Elevation of Bottom of Younger Bay Mud	PLATE 7
Muni Site	Oct. 2005
SFPUC ERP Power Plant	SF05019



LEGEND:

BAY MUD THICKNESS CONTOUR (FEET) 2 FOOT INTERVALS (APPROXIMATE BASED ON INTERPOLATION BETWEEN BORING OBSERVATIONS)

B-8 ● BORING LOCATION

(24) THICKNESS OF YOUNGER BAY MUD OBSERVED IN BORING

NOTE:
Base Map provided by BP POWER, Inc.
Reference: "MUNI SITE-PLOT PLAN-3 UNITS SIMPLE CYCLE",
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DRAWING NO. C1, Preliminary Issue

60' 30' 0 60' 120'

SCALE: 1" = 60'

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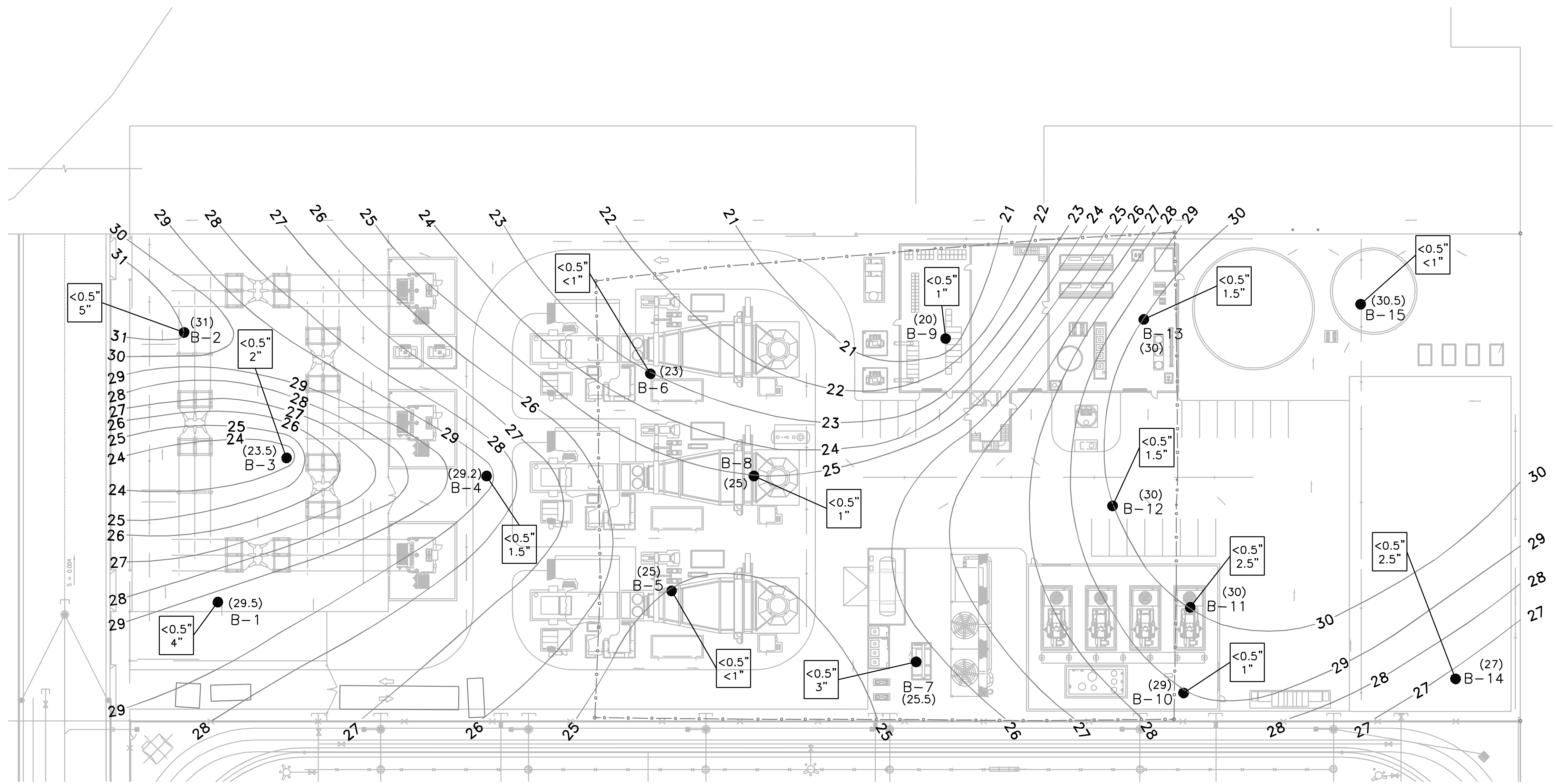
Thickness of Younger Bay Mud	PLATE 8
Muni Site	Oct. 2005
SFPUC ERP Power Plant	SF05019



LEGEND:

<0.5" 2" — ESTIMATED SETTLEMENT OF UNSATURATED FILL ABOVE THE GROUNDWATER TABLE (INCHES) / ESTIMATE OF SETTLEMENT OF SATURATED FILL REACHING LIQUEFACTION (INCHES)

26 — THICKNESS OF ARTIFICIAL FILL CONTOUR IN FEET (APPROXIMATE BASED ON INTERPOLATION BETWEEN BORING OBSERVATIONS)

B-8 ● BORING LOCATION

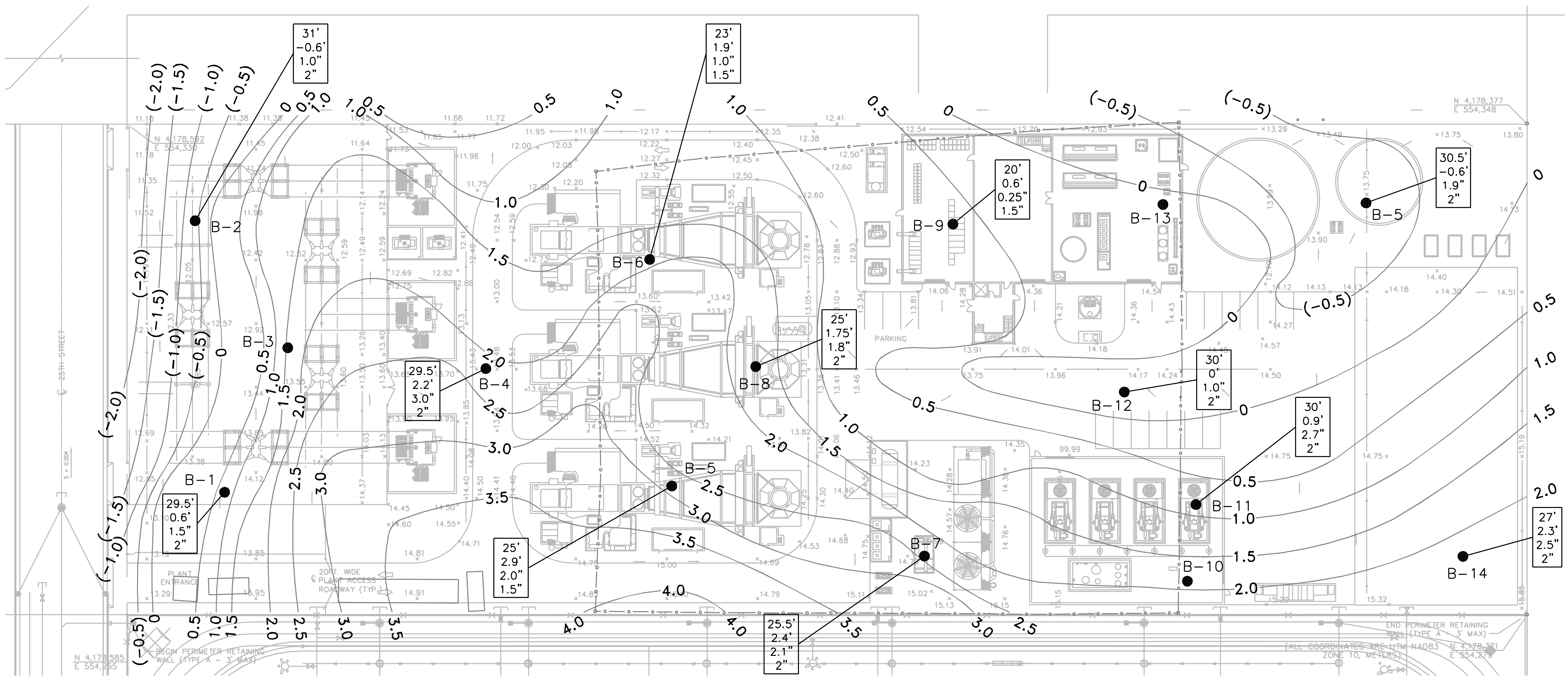
(30) THICKNESS OF ARTIFICIAL FILL (FEET) OBSERVED IN BORING

NOTE:
Base Map provided by BP POWER, Inc.
Reference: "MUNI SITE-PLOT PLAN-3 UNITS SIMPLE CYCLE",
SFERP, San Francisco, California
DRAWING NO. C1, Preliminary Issue

60' 30' 0 60' 120'

SCALE: 1" = 60'

GEOTECHNICAL CONSULTANTS, INC. 500 Sansome Street, Suite 402 San Francisco, CA 94111	Seismically Induced Settlements	PLATE 9
	Muni Site	Oct. 2005
	SFPUC ERP Power Plant	SF05019



NOTES:

1. Settlements shown are for consolidation of younger bay mud resulting from loads imposed by existing and proposed new fill. Consolidation settlement of fill and soils underlying the younger bay mud are considered negligible.
2. Secondary consolidation settlements shown are estimated for a period pf 50 years after completion of primary consolodation settlement.
3. Base Map provided by BP POWER, Inc. Reference: "MUNI SITE-PLOT PLAN-3 UNITS SIMPLE CYCLE", SFERP, San Francisco, California DRAWING NO. C1, Preliminary Issue

LEGEND:

● BORING LOCATION

25.5' — THICKNESS OF EXISTING FILL (FEET)
2.4' — THICKNESS OF NEW PROPOSED FILL (FEET)
2.1" — ESTIMATED CONSOLIDATION SETTLEMENT FROM NEW AND EXISTING FILL (INCHES)
2" — ESTIMATED SECONDARY CONSOLIDATION SETTLEMENT (INCHES)

× 14.75 PROPOSED DESIGN GRADE SPOT ELEVATION (FEET, NAVD 1988 DATUM)

1.5 THICKNESS OF NEW FILL CONTOUR, IN FEET (APPROXIMATE BASED ON SITE SURVEY TOPOGRAPHY AND PROPOSED DESIGN GRADE SPOT ELEVATIONS PROVIDED BY PB POWER, INC.

(-0.5) THICKNESS OF CUT CONTOUR, IN FEET (APPROXIMATE BASED ON SITE SURVEY TOPOGRAPHY AND DESIGN GRADE SPOT ELEVATIONS PROVIDED BY PB POWER, INC.

N

60' 30' 0 60' 120'

SCALE: 1" = 60'

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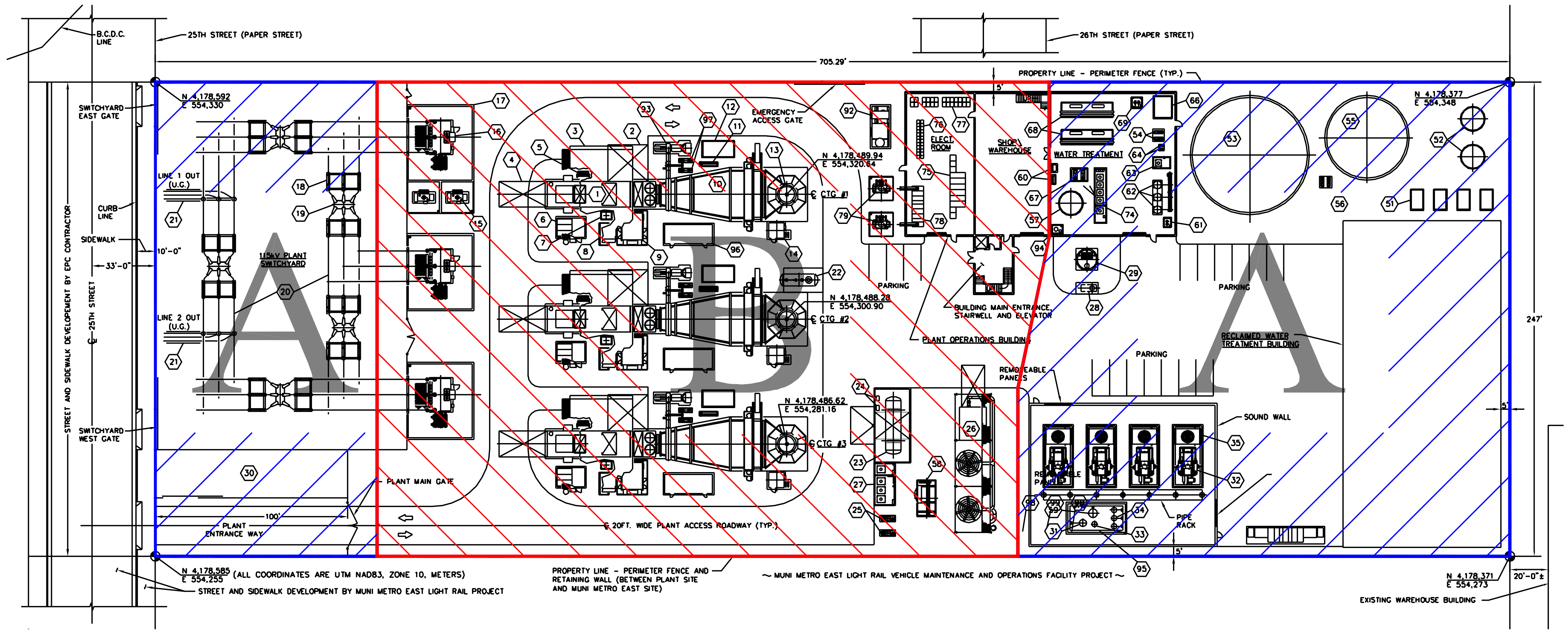
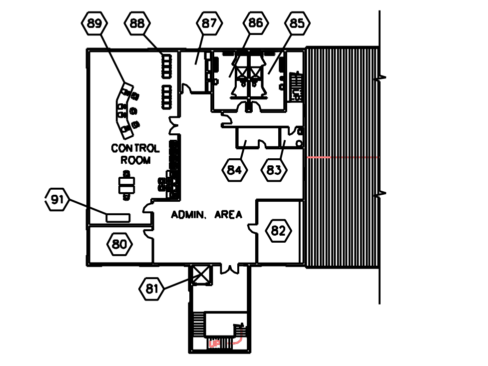
Settlement Induced by New and Existing Fill	PLATE 10
Muni Site	Oct. 2005
SFPUC ERP Power Plant	SF05019



PLOT PLAN



PLANT OPERATIONS BUILDING
2ND FLOOR – PLAN

LEGEND:

TYPICAL FOR THREE UNITS:

1. LM6000 COMBUSTION TURBINE GENERATOR
2. TURBINE REMOVAL/MAINTENANCE AREA
3. CTG AIR INTAKE FILTER SYSTEM
4. GENERATOR ROTOR REMOVAL AREA
5. CTG FIRE PROTECTION SKID
6. GENERATOR BREAKER SWITCHGEAR
7. SPRINT SYSTEM SKID
8. NOx WATER INJECTION SKID
9. AUXILIARY SKID
10. SCR/CO CATALYST SYSTEM
11. AMMONIA FLOW BALANCE SKID
12. AMMONIA VAPORIZATION SKID
13. STACK
14. CEMS

15. 5KV AUXILIARY TRANSFORMER (TYP. 2)
16. 13.8kV/115KV GSU (TYP. 3)
17. FIRE/BLAST WALL (TYP.)
18. 115kV SWITCH (TYP. 10)
19. 115 kV BREAKER (TYP.5)
20. SWITCHYARD BUS WORK
21. 115kV DUCT BANK
22. TURBINE WASH WATER DRAIN TANK (UG)
23. AQUEOUS AMMONIA STORAGE TANK
24. AQUEOUS AMMONIA FORWARDING PUMPS
25. AUXILIARY COOLING PUMPS
26. CHILLER/COOLING TOWER PACKAGE
27. COOLING TOWER CHEMICAL SYSTEM
28. OIL/WATER SEPARATOR (UG)
29. WASTE WATER SUMP AND LIFT STATION
30. 100'x25' PG&E GAS METERING STATION
31. NATURAL GAS INLET SCRUBBER
32. FUEL GAS COMPRESSOR (TYP. 4)
33. HYDROCARBON DRAIN TANK
34. DISCHARGE FILTER SCRUBBER (TYP. 2)
35. FUEL GAS COOLING RADIATOR (TYP.4)
36.–50. (HOLD)
51. ODOR CONTROL FANS (TYP. 4)
52. ODOR CONTROL SCRUBBERS
53. TREATED WATER STORAGE TANK
54. TREATED WATER PUMPS
55. DI WATER STORAGE TANK
56. DI WATER PUMPS
57. RO BREAK TANK
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63. SODIUM HYPOCHLORITE METERING SYSTEM
64. DOMESTIC NON-POTABLE WATER PUMPS
65. (NOT USED)
66. RO CLEAN IN PLACE SKID
67. 2nd PASS RO FEED PUMP SKID
68. RO TRAINS
69. RO CARTRIDGE FILTER SKID
70. (NOT USED)
71. (NOT USED)
72. (NOT USED)
73. (NOT USED)
74. DI WATER CHEMICAL METERING SYSTEM
75. 5kV SWITCHGEAR
76. 480V MCC'S
77. BATTERIES
78. 480V SWITCHGEAR
79. 480V STATION SERVICE TRANSFORMERS
80. PRIVATE OFFICE
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94. TEMPERED WATER SKID
95. FUEL GAS CONDITIONING SKID
96. POWER AND CONTROL CAB (PCC) (TYP. EA. CTG)
97. DAMPER SEAL AIR FANS (TYP. AT EACH SCR)
98. 480V DRY TYPE TRANSFORMER
99. 480V SWITCHGEAR, RECLAIMED WATER AREA

RECOMMENDED PILE ELEVATIONS FOR
14"–SQUARE PRESTRESSED PRECAST CONCRETE PILES

ALLOWABLE CAPACITY IN VERIFICAL COMPRESSION (tons)	PILE TIP ELEVATION (feet)[1]		ULTIMATE CAPACITY IN SHORT–TERM UPLIFT (tons)
	AREA A	AREA B	
100	–75	–95	100
125	–95	–105	125

[1] ELEVATIONS ARE WITH RESPECT TO NAVD 88 DATUM

KEY:

A — PILE DESIGN AREA A

B — PILE DESIGN AREA B

REFERENCE:
"Muni Site Plot Plan" Drawing No. G1.2,
San Francisco Electric Reliability Project,
by PB Power, Inc.

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Pile Design Areas A and B	PLATE 11
Muni Site	Oct. 2005
SFPUC ERP Power Plant	SF05019

APPENDIX A –
SUPPORTING GEOTECHNICAL DATA

APPENDIX A
SUPPORTING GEOTECHNICAL DATA

SUBSURFACE EXPLORATION

Subsurface exploration for our geotechnical investigation for the Muni Site Power Plant took place between July 23 and August 2, 2005, and consisted of drilling fifteen rotary wash borings, B-1 through B-15. Because of heavy equipment and truck traffic, borings in the existing batch plant area, B-1 to B-4, were drilled on two Saturdays. A "trash barrel" 8-inch coring bit was used to progress all of the borings to approximately 15 feet, after which rotary wash drilling techniques were used. Steel casing was set into the Younger Bay Mud in all the borings to prevent caving and loss of drilling fluid into the artificial fill. Two borings, B-10 and B-11 were cased with three-inch PVC pipe for geophysics testing. Upon completion of these borings, the top five feet of casing was drilled out, and the borings were backfilled with cement grout. All of the other borings were backfilled with cement grout. The following table shows the depths of the borings.

TABLE A-1 – BORING DEPTHS

Boring	Depth (feet)
B-1	100.5
B-2	101.5
B-3	32.5
B-4	168.2
B-5	100.5
B-6	100.5
B-7	101.0
B-8	101.5
B-9	100.9
B-10	31.5
B-11	101.5
B-12	101.5
B-13	33.0
B-14	101.5
B-15	150.0

Locations of the borings are shown on Plate 2. Logs of the borings are presented as Plates A-1.1 through A-1.3.

The stratification lines shown on the boring logs represent the approximate boundaries between soil types; the actual transition may be gradual. The boring locations were measured in the field with a measuring wheel from the fence lines. The site was surveyed by the SFPUC and they provided a topographical map based on that survey. We estimated our boring elevations from the topographic map. The locations and elevations of borings should be considered accurate only to the degree implied by the method used.

We experienced very difficult drilling conditions at the project site. The artificial fill is comprised of mostly debris, including concrete, metal, glass, wood, and bricks, in a sand and gravel matrix. Because of the nature of the artificial fill, an eight-inch coring barrel, or "trash barrel", was needed to drill through the fill to depths of approximately 15 feet, after which rotary wash drilling methods were used. Casing was driven approximately 25 to 30 feet, or into the top of the Younger Bay Mud layer to prevent the hole from caving and to maintain circulation of drilling fluid. In several borings, obstructions were encountered where drilling about one to two feet through the obstruction took two to three hours. Table A-2 – Debris Encountered During Subsurface Exploration summarizes the depths of notable debris, the time it required to drill through the artificial fill, and the nature of the debris. It should be noted that just the most dominate or notable layers are listed in the table. Also, the fill is heterogeneous in nature; therefore, significant obstructions should be expected when drilling or excavating in other locations on the project site.

TABLE A-2 – DEBRIS ENCOUNTERED DURING SUBSURFACE EXPLORATION

Boring Number	Approximate Time to Drill Through Artificial Fill (hours)	Depth of Notable Debris (feet)	Type of Debris	Comments
B-1	4	5 to 6	concrete slab	-
B-2	5	25 to 27	boulder/cobble	-
		4 to 31	cobbles, 3 to 5 inches	-
B-3	2	6	cobbles up to 6 inches	-
B-4	4	0 to 15	brick, tile, wood	-
B-5	7	2.5 to 7	concrete slab	very hard drilling
		7 to 25	glass, brick, cobbles	-
		13 to 15	metal	very hard drilling, wore out two "trash barrel" bits
B-6	4	0 to 19	concrete, metal, glass, bricks	abundant bricks from 15 to 19 feet
B-7	3	0 to 16	brick, concrete	-
		16 to 17	concrete slab	-
B-8	5	2	steel rebar, concrete	-
		9	concrete	bent steel casing
		10 to 25	brick, concrete	-
B-9	3	15 to 18	metal	very hard drilling
B-10	7	9 to 14	brick	abundant
		14 to 25	glass, brick, concrete, metal, tile, cobbles	metal at 22 feet
B-11	4	9 to 20	brick, concrete, metal, tile	abundant brick at 17 feet
		12 to 14	concrete slab	-
		20 to 25	wood	abundant
B-12	5	0 to 20	brick, glass, metal	-
		20 to 30	concrete	-
B-13	4	5 to 15	brick, glass, concrete	-
		20 to 22	wood	-
		25 to 27	brick, metal	-
B-14	5	3 to 5	wood, concrete	-
		12 to 13	concrete slab	very hard drilling
		13 to 25	concrete, glass, brick, wire	very hard drilling
B-15	6	9 to 30	brick and wood	-

SOIL SAMPLING METHODS

Soil sampling methods used during the exploration program were Standard Penetration Tests (SPT), a 2.5-inch diameter split barrel sampler, and Shelby tubes.

Standard penetration tests (SPT's) were performed by using a 2-inch outside diameter, 1.38-inch inside diameter steel sampler. The sampler was driven by repeatedly dropping a 140-pound hammer approximately 30 inches onto the sampling rod to which the sampler was attached. The number of blows required to drive the sampler the last 12 inches of a total 18-inch interval is referred to as the standard penetration test blow count or N-value, and is recorded on the drill hole logs.

A split barrel sampler was driven a total of 18 inches or until refusal per ASTM D1586. The soil was driven into three six-inch long, 2.5-inch inside diameter brass liners and the sampler shoe. The sampler was driven by repeatedly dropping a 140-pound hammer approximately 30 inches into the drill rod to which the sampler was attached. The number of blows required to drive the sampler the last 12 inches of a total of 18-inch interval is referred to as the blow count and is recorded on the boring logs. Blow counts were recorded for the purpose of estimating relative soil densities.

Samples were collected within the younger bay mud by using thin walled Shelby tubes measuring three inches in diameter and three feet in length.

LABORATORY TESTING

Laboratory tests were performed on representative soil samples in order to define the engineering properties of the earth materials. Testing procedures followed accepted practice where possible. Where ASTM Standards were used, the latest edition or revision for each test procedure was employed.

MOISTURE AND DENSITY DETERMINATIONS

Moisture content and dry density determinations were performed on representative undisturbed samples from fourteen of the fifteen borings to evaluate the natural water content and dry density of the soils encountered. The results are presented on the boring logs.

GRAIN SIZE DISTRIBUTION DATA

Grain-size distribution tests were conducted on 32 samples from all borings, except B-3. The tests were performed in accordance with Standard Test Method ASTM D422 - Standard Method for Particle-Size Analysis of Soils. The results of the tests are included in this Appendix.

ATTERBERG LIMITS

Atterberg limits were performed on 16 samples from borings B-1, B-2, B-4, B-6, B-7, B-8, B-9, B-11, B-12, B-14, and B-15. Testing was performed in accordance with ASTM D4218 - Liquid Limit, Plastic Limit, and Plasticity Index of Soils. Results of these tests are presented on the boring logs.

CONSOLIDATION TESTS

Three consolidations tests were performed on 12 samples from B-1, B-2, B-4, B-5, B-6, B-7, B-8, B-9, B-11, B-12, B-14, and B-15. Testing was performed in accordance with Standard Test Method ASTM D2435. The results of the tests are included in this Appendix.

UNCONSOLIDATED UNDRAINED TRIAXIAL TESTS (UU)

Unconsolidated undrained triaxial tests were performed on 9 samples from B-1, B-4, B-5, B-6, B-7, B-9, B-11, B-12, and B-15. Testing was performed in accordance with Standard Test Method ASTM D2850 – Unconsolidated Undrained Triaxial Test on Cohesive Soils. The results of the tests are included in this Appendix.

UNCONFINED COMPRESSION TESTS (UCS)

Unconfined compression tests were performed on 4 samples from B-7, B-9, B-11, and B-15. Testing was performed in accordance with Standard Test Method ASTM D2166 – Standard Test Method for Unconfined Compressive Strength of Cohesive Soil. The results of these tests are presented on the boring logs.

R-VALUE TESTING

R-value testing was performed on three samples from B-2, B-4 and B-7. Testing was performed in accordance with Cal-Test 301 procedures. The resistance values are summarized in the following table.

TABLE A-3 – R-VALUE TESTING SUMMARY

Boring	Depth (feet)	Resistance Value
B-2	2 to 5	80
B-4	1 to 4	85
B-7	2 to 5	45

CORROSION TESTING

Corrosion testing was performed on 4 samples from B-6, B-8, B-11, and B-12. Testing was performed in accordance with Cal-Tests 643, 532, 422, and 417; and ASTM D458 procedures. The results are summarized in the following table.

TABLE A-4 – CORROSION TESTING SUMMARY

Boring	Depth (feet)	Resistivity (ohm-cm)		pH	Sulfates (ppm)	Chlorides (ppm)
		As received	Saturated			
B-6	14-15.5	1,653	1,318	10.79	1,900	530
B-8	45.5-46	180	159	8.15	3,300	3,140
B-11	5.5-6	49,173	6,205	8.72	290	72
B-12	65-66.5	646	555	8.73	80	452

LOG OF DRILL HOLE

JOB NO.: SF05019 | LOGGED BY: A. Killeen | DRILL HOLE NO.: B-1
PROJECT: Muni Power Plant | CHECKED BY: J. Seibold | DRILLING DATE: 7/23/2005
LOCATION: Lot between east ends of 25th St. & Cesar Chavez | ELEVATION: 12.0 feet
DRILLING METHOD: Rotary Wash, 5-inch dia. | DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							"ARTIFICIAL FILL (af)" 3-inches SANDY GRAVEL (GP) over 10-inch CONCRETE slab.. SANDY GRAVEL (GW), gray, moist, loose, well graded angular gravel to 1-inch.						
5		50/5"					6-inch buried concrete slab.						
							2½-inch serpentinite fragments. CLAYEY GRAVEL (GC), gray, moist. loose, decomposed serpentinite fragments up to 3-inches in clay and gravel matrix.						
10		7											
15		15											
		14					WELL GRADED GRAVEL with SILT and SAND (GW-GM), gray to brown, wet, loose, angular decomposed serpentinite fragments to 1½-inches, chert clasts.	104	12				GS
20													
25		4					POORLY GRADED GRAVEL with CLAY and SAND (GP-GC), greenish gray, wet, loose.						GS
30		6					"YOUNGER BAY MUD (Qybm)" FAT CLAY (CH), dark greenish gray, moist, medium stiff, serpentinite fragments to ¾-inch, medium plasticity.						
35		120 psi		0.5			No serpentinite fragments. Trace shell fragments, high plasticity.	74	45				UU, C
40		3											
45		6	0.45	0.4			Decreasing shell fragments.						
50		15					"UPPER LAYERED SEDIMENTS (Quls)" CLAYEY SAND (SC), dark greenish gray, moist, medium dense, very fine grained sand, high plasticity clay.						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LOG OF DRILL HOLE

JOB NO.: SF05019 | LOGGED BY: A. Killeen | DRILL HOLE NO.: B-1
PROJECT: Muni Power Plant | CHECKED BY: J. Seibold | DRILLING DATE: 7/23/2005
LOCATION: Lot between east ends of 25th St. & Cesar Chavez | ELEVATION: 12.0 feet
DRILLING METHOD: Rotary Wash, 5-inch dia. | DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
		20		1.4			LEAN CLAY (CL), greenish gray, moist, stiff, medium plasticity, trace to minor fine grained sand and silt.						
							Increasing sand content.						
60		50					POORLY GRADED SAND (SP), light brown, moist, very dense, fine grained sand, trace to minor silt and clay, trace chert fragments.						
65		29					CLAYEY SAND (SC), light brown, moist, medium dense, fine grained sand, trace silt.			29	17		GS
70		100/11"					POORLY GRADED SAND (SP), light to dark brown, wet, very dense, fine grained sand, trace silt.						
75		77											
80		15	0.45	1.9			LEAN CLAY (CL), light brown with reddish brown mottles, moist, stiff, minor fine grained sand, trace dark brown chert fragments, trace decomposed organics.						
85													
90		6	0.45	0.7			"OLDER BAY MUD (Qobm)" FAT CLAY (CH), greenish gray, moist, medium stiff, high plasticity.						
95							Lens of light brown sand.						
100		22		1.6			"UPPER LAYERED SEDIMENTS (Quls)" SILTY CLAY (CL), greenish gray, moist, very stiff, low plasticity.						
105							1) Bottom of boring at 100.5 feet. 2) Groundwater encountered at 10.5 feet. 3) Boring backfilled with cement grout.						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: Rotary Wash, 5-inch dia.

LOGGED BY: J. Seibold
CHECKED BY: A. Killeen

DRILL HOLE NO.: B-2
DRILLING DATE: 7/30/2005
ELEVATION: ~13.5
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							"ARTIFICIAL FILL (af)" 9-inches concrete cement.						
		Bulk Sample					SILTY SAND with GRAVEL (SM), dark yellowish brown, damp, mixed gravel - predominantly fine to medium grained, subrounded to subangular clasts, occasional cobble sized clasts (3- to 5-inches in diameter). Grading to light olive gray with orange mottling, serpentinite derived fill, abundant cobbles. Loose to medium dense.						R-Value GS
5													
		11											
10		11					Wet.						
							WELL GRADED GRAVEL (GW-GM) with SAND and SILT, grayish green to greenish black, wet, loose, well graded serpentinite gravel, serpentinite derived sand and silt, minor cobbles. Minor coarse angular gravel and and cobbles to 3-inches.						
15		9											
		13						112	13				GS
20		4					Trace clayey fines, minor greenstone clasts.						
							Minor clay.						
25							Approximately 1-foot thick boulder.						
		9											
30		4											
							"YOUNGER BAY MUD (Qybm)" FAT CLAY (CH), dark greenish gray, moist, soft, scattered shell fragments.						
35		125 psi											
				0.55						73	29		C
40		3	0.24	0.50									
45		150 psi											
				0.75			Minor fine grained sand.						
							Increasing sand content.						
							SANDY CLAY (CL), dark greenish gray, wet, soft to medium stiff,approximately 35% fine grained sand in medium plasticity silty clay.						
50		8					Approximately 6-inch layer of silty sand.						
							FAT CLAY (CH) with SAND, dark greenish gray, moist, fine grained sand.						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: Rotary Wash, 5-inch dia.

LOGGED BY: J. Seibold
CHECKED BY: A. Killeen

DRILL HOLE NO.: B-2
DRILLING DATE: 7/30/2005
ELEVATION: ~13.5
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
		66		4.75± >4.5			"UPPER LAYERED SEDIMENTS (Quls)" SILTY CLAY (CL) with SAND, dark greenish gray, moist, very stiff, medium plasticity silty clay with approximately 5% fine grained sand.						
							SANDY CLAY (CL), medium yellowish brown with orange mottling, moist, very stiff, low plasticity clay with approximately 25% fine grained sand.						
60		13		2.75			Dusky yellow to light olive gray with some dark orange mottling, stiff.						
							CLAYEY SAND (SC), medium yellowish brown, moist, dense to very dense, fine grained sand with approximately 30% low plasticity clay and minor fine angular gravel (chert clasts).						
65		56						111	18				GS
70		32					Interbedded SILTY SAND (SM) and CLAYEY SILT (ML), olive gray sand and dark yellowish orange silt, wet, stiff to dense, fine to very fine grained sand, low plasticity fines.						
75		65					SILTY SAND (SM) moderate to dark yellowish brown, moist, very dense, fine grained sand, approximately 20% fines, occasional very thin silt lenses.						
80		23		2.10			'OLDER BAY MUD (Qobm)" FAT CLAY (CH) with SILT, grayish green to dark greenish gray, moist stiff, high plasticity clay, approximately 5-10% silt, trace shell fragments.						
85													
90		0		0.50			Soft, no shell fragments, decreased silt content.						
							Approximately 1-foot thick layer of FAT CLAY with SAND, dark greenish gray, moist, soft, approximately 10% fine grained sand.						
95													
							Increasing sand, trace fine gravel.						
100		22					"UPPER LAYERED SEDIMENTS (Quls)" SANDY CLAY/CLAYEY SAND (CL/SC), blue green, moist to wet, stiff to medium dense, fine grained sand, medium plasticity clay, approximately 5% fine to medium angular gravel.						
105							1) Bottom of boring at 101.5 feet. 2) Groundwater measured at 9.35 feet. 3) Boring backfilled with cement grout.						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: Rotary Wash, 5-inch dia.

LOGGED BY: D. van Hoff
CHECKED BY: A. Killeen

DRILL HOLE NO.: B-3
DRILLING DATE: 7/23/2005
ELEVATION: 13.5 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							"ARTIFICIAL FILL (af)" 18-inches PORTLAND CEMENT CONCRETE SLAB. GRAVELLY SAND (SP), brown with black staining, moist, medium dense. SANDY GRAVEL with CLAY (GP-GC), brown, moist, medium dense, minor clay.						
5		46					At 3.5 feet - very dark brown, wet, rounded to angular gravel to 2-inches. At 6.5 feet - dark greenish gray, becoming silty,						
10		21					CLAYEY GRAVEL (GC), dark greenish gray, wet, medium dense, serpentinite gravel clasts to 2-inches. SANDY GRAVEL (GP), dark brown, wet, medium dense. At 11 feet - dark greenish gray, becoming silty, serpentinite gravel clasts to 3-inches.						
15		6					POORLY GRADED GRAVEL (GP), gray, red, and dark greenish gray, wet, loose, angular chert and serpentinite clasts to 1½-inches.						
20		17					CLAYEY GRAVEL (GC), very dark brown, wet, medium dense, abundant wood and brick debris.						
25		1					"YOUNGER BAY MUD (Qybm)" FAT CLAY (CH), dark gray, wet, soft, silty.						
30		125 psi											
35							1) Bottom of boring at 32.5 feet. 2) Groundwater measured at 10.2 feet. 3) Boring backfilled with cement grout.						
40													
45													
50													

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash

LOGGED BY: J. Seibold
CHECKED BY: A. Killeen

DRILL HOLE NO.: B-4
DRILLING DATE: 7/23/2005
ELEVATION: 10.5 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							"ARTIFICIAL FILL (af)"						
		Bulk Sample					POORLY GRADED SAND with SILT and GRAVEL (SP-SM), light olive gray, damp, fine to medium grained sand, rounded to subrounded gravel to 1½-inch.						R-Value GS
5		40					Medium dense, fine to coarse grained sand, zones of decomposed serpentinite, trace brick and ceramic tile fragments.						
10		10					Loose to medium dense, wood and brick fragments, minor black staining.						GS
15		9					CLAYEY SAND (SC) with GRAVEL, light olive gray, damp, loose, predominantly decomposed serpentinite with angular gravel clasts to 1½-inches, serpentinite and greenstone clasts.						
20		13					Medium dense.						
25		19					WELL GRADED GRAVEL with SILT and SAND (GW-GM), light olive gray, damp, loose, serpentinite gravel.						GS
							3-inch cobble in cuttings.						
30		7					"YOUNGER BAY MUD (Qybm)"						
				0.20			FAT CLAY (CH), dark greenish gray, moist, very soft to soft, high plasticity clay with silt and scattered shell fragments.						
35		150 psi											
				0.55				71	49				UU, C
40		4	0.25	0.60			Cobble. Soft to medium stiff, increasing shell fragments.						
45		75 psi					No apparent shell fragments.						
		6	0.28										

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019 | LOGGED BY: J. Seibold | DRILL HOLE NO.: B-4
PROJECT: Muni Power Plant | CHECKED BY: A. Killeen | DRILLING DATE: 7/23/2005
LOCATION: Lot between east ends of 25th St. & Cesar Chavez | ELEVATION: 10.5 feet
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash | DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
				0.55									
55		0 psi											
			0.31										
							SANDY CLAY (CH), dark gray, moist, soft, high plasticity, very fine grained sand, trace organics at 60.5 feet.						
60		0	0.15										
65		24					"UPPER LAYERED SEDIMENTS (Quls)" SANDY CLAY (CL), greenish gray, moist, very stiff, very fine to fine grained sand.			28	20		GS
							POORLY GRADED SAND (SP), yellowish gray grading to dark yellowish orange, moist, very dense, fine grained sand, approximately 2-5% silt.						
70		50/6"											
							POORLY GRADED SAND with SILT (SP-SM), moderate yellowish brown, wet, very dense, fine grained sand, approximately 10% silt.						
75		50/6"											
80							"OLDER BAY MUD (Qobm)" SILTY CLAY (CH), grayish blue green with minor grayish orange mottling, moist, medium stiff to stiff, high plasticity clay, approximately 15% silt.						
		15		1.5				62	65	93	33		
85													
90		0		0.55			Medium stiff, medium blue gray, no mottling.						
95													
		0					CLAYEY SILT (ML), olive black, wet, soft, scattered wood fragments.						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash

LOGGED BY: J. Seibold
CHECKED BY: A. Killeen

DRILL HOLE NO.: B-4
DRILLING DATE: 7/23/2005
ELEVATION: 10.5 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
105													
							"UPPER LAYERED SEDIMENTS (Quls)" POORLY GRADED SAND with SILT (SP-SM), yellowish to light olive gray, wet, very dense, fine grained sand.						
		50/6"											
110													
115													
							"OLDER BAY MUD (Qobm)" FAT CLAY (CH), dark greenish gray, moist, medium stiff, high plasticity clay.						
		0											
120				1.1									
125													
		7											
130													
135													
140													
145							"LOWER LAYERED SEDIMENTS (Qlls)" SANDY CLAY (CL) with GRAVEL, grayish blue green, moist, very stiff to hard, low plasticity silty clay, approximately 20% very fine grained sand and 20% medium angular gravel (shale fragments).						
		71											

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash

LOGGED BY: J. Seibold
CHECKED BY: A. Killeen

DRILL HOLE NO.: B-4
DRILLING DATE: 7/23/2005
ELEVATION: 10.5 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
155													
							FAT CLAY (CH), pale green, moist.						
160													
							SANDY CLAY (CL) with GRAVEL, pale green???, moist.						
165													
		100/2"					"FRANCISCAN COMPLEX (KJf)" SHALE (R), dark gray to black, moderately strong, fractured.						
170													
175													
180													
185													
190													
195													

1) Bottom on boring at 168 feet and 2-inches.
2) Groundwater measured at 10 feet.
3) Boring backfilled with cement grout.

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash
LOGGED BY: J. Seibold & D. van Hoff
CHECKED BY: A. Killeen
DRILL HOLE NO.: B-5
DRILLING DATE: 7/27/2005
ELEVATION: 11.0 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							"ARTIFICIAL FILL (af)" SANDY GRAVEL with SILT (GW-GM), pale yellowish brown, damp, fine to coarse gravel, rounded to angular clasts.						
							Concrete slabs/blocks.						
5							SANDY GRAVEL (GP), moderate yellowish brown, damp, dense, fine gravel with minor rounded medium to coarse clasts, fine grained sand, trace to minor silt, trace glass and brick fragments.						
		49					MIXED FILL DEBRIS, mixed gravel, cobbles, boulders, brick, concrete and glass fragments in a sand and silt matrix with clayey pockets and zones. Cored through approximately 9-inch "cobblestone" boulder.						
10		17											GS
							At 13 to 14 feet - metal debris. Hard obstruction at 14 feet (metal), difficult drilling.						
15													
20													
25		7	0.25				"YOUNGER BAY MUD (Qybm)" FAT CLAY (CH), dark greenish gray, moist, soft, scattered shell fragments.						
30		75 psi	0.39	0.7			Dark gray, wet.						
35		7	0.42	0.4									
40		100 psi	0.44	0.8			Fewer shell fragments.	68	52				UU, C
45		2	0.45	0.5									
50		150 psi					Lense of dark gray Sandy Clayey Silt, wet, medium stiff.						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash

LOGGED BY: J. Seibold & D. van Hoff
CHECKED BY: A. Killeen

DRILL HOLE NO.: B-5
DRILLING DATE: 7/27/2005
ELEVATION: 11.0 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
		0					Dark gray, wet, very soft, minor shell fragments.						
60		5	0.57	1.0			Soft.						
65		75					"UPPER LAYERED SEDIMENTS (Quls)" SILTY SAND (SM), grayish olive green, wet, very dense.	113	19				
70		88					POORLY GRADED SAND (SP), dark yellowish brown, wet, very dense, fine to medium grained sand.						GS
75		51					CLAY (CL), brown with orange mottling, wet, hard.						
80		10	0.62	1.2			"OLDER BAY MUD (Qobm)" FAT CLAY (CH), mottled dark gray and brown, wet, stiff.						
85													
90		100 psi	0.40	1.3									
95													
100		50/6"					"UPPER LAYERED SEDIMENTS (Quls)" GRAVELLY SAND (SP), orange brown, wet, very dense.						
105							1) Bottom of boring at 100.5 feet. 2) Groundwater measured at 9.4 feet on 7/27/05. 3) Boring backfilled with cement grout.						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

PLATE A-1.5

LOG OF DRILL HOLE

JOB NO.: SF05019 | LOGGED BY: A. Killeen | DRILL HOLE NO.: B-6
PROJECT: Muni Power Plant | CHECKED BY: J. Seibold | DRILLING DATE: 7/27/2005
LOCATION: Lot between east ends of 25th St. & Cesar Chavez | ELEVATION: 11.0 feet
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash | DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							"ARTIFICIAL FILL (af)" SANDY GRAVEL (GW), bluish gray to black, dry, loose, serpentinite fragments to 2-inches, concrete, metal, glass, and brick fragments.						
5		50/2"											
10		31											
15		51					Lens of Poorly Graded Sand. Abundant bricks and brick fragments.						Corr
							Wood fragments.						
20		15					POORLY GRADED GRAVEL with SAND (GP), black, wet, medium dense, fine grained sand, abundant wood, black oily substance throughout to 24 feet.						GS
							"YOUNGER BAY MUD (Qybm)" FAT CLAY (CH), dark greenish gray, moist, medium stiff, high plasticity fines, shell fragments.						
25		9											
30		50 psi					Slight H_2S odor.						
35		0					Decreasing shell fragments.						
40													
				0.3									
45		8		0.4									
50		125 psi		0.8				68	52	75	30		UU, C
		10					"UPPER LAYERED SEDIMENTS (Quls)"						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash

LOGGED BY: A. Killeen
CHECKED BY: J. Seibold

DRILL HOLE NO.: B-6
DRILLING DATE: 7/27/2005
ELEVATION: 11.0 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							CLAYEY SAND (SC) with SILT, dark greenish gray, wet, loose, fine grained sand, high plasticity clay.						
							SANDY CLAY (CL) with SILT, greenish gray, wet, soft, medium plasticity clay, fine grained sand.						
60		3		0.3									
65		11					Increasing clay content, stiff, trace organics.			30	15		
							CLAYEY SAND (SC), greenish gray, moist, medium dense, fine grained, trace organics.						
70		20											GS
75		46					POORLY GRADED SAND with SILT (SP-SM), brown, moist, dense, fine grained sand.						
							LEAN CLAY (CL/CH), yellowish brown with greenish gray and reddish brown mottles, moist, stiff, reddish brown pockets.						
80		14		1.7									
85													
							"OLDER BAY MUD (Qobm)" FAT CLAY (CH), greenish gray, moist, stiff.						
90		100 psi											
95													
							"UPPER LAYERED SEDIMENTS (Quls)" SANDY SILT (ML), greenish gray, moist, very stiff, low plasticity fines.						
100		85					POORLY GRADED SAND with SILT (SP-SM), brown, moist, very dense, fine grained sand.						
105													

1) Bottom of boring at 100.5 feet.
2) Groundwater measured at 9.3 feet.
3) Boring backfilled with cement grout.

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash

LOGGED BY: A. Killeen
CHECKED BY: J. Seibold

DRILL HOLE NO.: B-7
DRILLING DATE: 8/2/2005
ELEVATION: 11.5 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
0							"ARTIFICIAL FILL (af)" SILTY GRAVEL with SAND (GM), light brown to gray, dry, loose, angular gravel, serpentinite fragments, abundant concrete fragments with brick, fine grained sand.						
		Bulk Sample											R-Value GS
5		50/6"											
10		9					WELL GRADED SAND (SW) with GRAVEL, light brown, moist, fine grained sand, angular to flat gravel, some brick fragments.						
15							Approximately 1-foot thick concrete slab/block. Concrete fragments.						
20		16											
25		7					"YOUNGER BAY MUD (Qybm)" FAT CLAY (CH), dark greenish gray, moist, soft, high plasticity clay, trace shell fragments.						
30		75 psi		0.6				70	52	68	28		UU C
35		7					Abundant shell fragments, metal key?						
40		75 psi		0.5			Decreasing shell fragments.						
45		5		0.3									
50		15					Trace organics.						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash

LOGGED BY: A. Killeen
CHECKED BY: J. Seibold

DRILL HOLE NO.: B-7
DRILLING DATE: 8/2/2005
ELEVATION: 11.5 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
		22		1.6			Lenses of dark greenish gray poorly graded sand.						
60		31					"UPPER LAYERED SEDIMENTS (Quls)" SILTY SAND (SM), dark greenish gray, wet, dense, fine grained sand.						
65		85					Very dense. Dark greenish gray sand with lenses of yellowish brown clay.						
70		95					Decreasing silt content. At 70.5 feet - moderate yellowish brown. At 71.5 feet - dark reddish brown mottling.						
75		50/6"					POORLY GRADED SAND with SILT (SP-SM), dark greenish gray with moderate yellowish brown mottling, wet, dense, trace subrounded flat serpentinite fragments and red gravel.	105	21				GS
							"OLDER BAY MUD (Qobm)"						
80		150 psi					FAT CLAY (CH), greenish gray with moderate yellowish brown veining, moist , soft.	68	57	89	31	3394	
				1.2									
85													
90		150 psi											
				1.7									
95													
100		50/6"					"UPPER LAYERED SEDIMENTS (Quls)" POORLY GRADED SAND (SP), moderate yellowish brown, wet, very dense, fine grained sand, black sand grains throughout.						
105							1) Bottom of boring at 101 feet. 2) Groundwater measured at 11.5 feet on 8/02/05. 3) boring backfilled with cement grout.						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 6-inch Core Barrel and 5-inch dia. Rotary Wash

LOGGED BY: D. van Hoff & A. Killeen
CHECKED BY: J. Seibold

DRILL HOLE NO.: B-8
DRILLING DATE: 7/28 & 7/29/2005
ELEVATION: 11.5 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							"ARTIFICIAL FILL (af)" SILTY GRAVEL (GM), gray, damp. medium dense, concrete debris greater than 6-inches in size, minor steel rebar pieces.						
5													
		31					Less concrete and rebar debris, minor clay.						
							At 8 feet - approximately 1 foot thick layer of Gravely Clay (CL), brown, moist, stiff, with some metal debris.						
10		20					brown, rounded poorly graded 1- to 2-inch diameter gravel clasts, concrete debris up to 2-inches in size.						
							CLAYEY GRAVEL (GC), dark brown, wet, loose, minor brick and construction debris. At 12 feet - abundant red brick fragments. At 12 to 13 feet - abundant concrete debris, with concrete blocks/pieces greater than 6-inches. At 15 feet - abundant red brick fragments.						
15		47											
							WELL GRADED GRAVEL (GW), red and gray, wet, medium dense, coarse gravel, trace fines, red brick, concrete rubble, and gravel clasts from ¾- to 2½-inches in diameter.						
20		38						80	12				GS
25		7	0.5	0.8			"YOUNGER BAY MUD (Qybm)" FAT CLAY (CH),dark greenish gray, wet, medium stiff, scattered shell fragments.						
30		75 psi											
35		0		0.3									
40				0.5			Trace shell fragments.						
45		5		0.3									Corr
50		3											

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 6-inch Core Barrel and 5-inch dia. Rotary Wash
LOGGED BY: D. van Hoff & A. Killeen
CHECKED BY: J. Seibold
DRILL HOLE NO.: B-8
DRILLING DATE: 7/28 & 7/29/2005
ELEVATION: 11.5 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
		125 psi		0.3									
60		8	0.24	0.7									
							"UPPER LAYERED SEDIMENTS (Quls)" SILTY SAND (SM), greenish gray, moist, medium dense, fine grained sand, trace organics.						
65		18											
							Very dense, trace very dark brown gravel to ¼-inch.						
70		99											GS
75		50/6"					Moderate yellowish brown to light brown.						
							"OLDER BAY MUD (Qobm)" FAT CLAY (CH), yellowish brown with reddish brown mottles, moist, very stiff, minor silt.						
80		22		1.7									
85							Dark greenish gray (5G 4/1).						
90		150 psi		1.5						75	25		C
95							Yellowish brown.						
100		68		3.7			"UPPER LAYERED SEDIMENTS (Quls)" POORLY GRADED SAND with SILT (SP-SM) and CLAY, brown with grains of black fine grained sand, moist, very dense.						
105							1) Bottom boring at 101.5 feet. 2) Groundwater measured at 12.3 feet on 7/28/05. 3) Boring backfilled with cement grout.						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash

LOGGED BY: A. Killeen & J. Seibold
CHECKED BY: D. van Hoff

DRILL HOLE NO.: B-9
DRILLING DATE: 7/29/ & 8/01/2005
ELEVATION: 12.5 feet
DATUM: NAVD 1988

DEPTH (FEET)	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
					"ARTIFICIAL FILL (af)" WELL GRADED SAND (SW) with GRAVEL, yellow brown, dry, loose, fine grained sand, angular gravel.						
					WELL GRADED GRAVEL (GW), dark brown to yellowish brown, dry, very dense, decomposed serpentinite clasts up to 2-inches, trace glass fragments, lenses of black hydrocarbon material.						
5	94										
					SILTY GRAVEL with SAND (GM), dusky brown, moist, medium dense, mixed gravel, fine to coarse subrounded to subangular gravel clasts, predominantly Franciscan derived clasts (serpentinite, greenstone, graywacke), approximately 40% well graded sand, minor crushed brick fragments.						
10	16										GS
	Bulk Sample										
15					Difficult drilling, piece(s) of metal from 15½ to 18 feet.						
	14										
20					"YOUNGER BAY MUD (Qybm)" FAT CLAY (CH), dark greenish gray, moist, soft, scattered shell fragments.						
	11		0.45								
25	150 psi				Sample disturbed by metal and wood fragments from falling slough.						
	100 psi										
30			0.45								C
35	5	0.18	0.55		Minor H_2S odor.						
40	100 psi				No H_2S odor.	67	54				UU
			0.65								
45	8	0.28	0.65								
50	5	0.24	0.65			65	58	73	28		
					Trace to minor sand, sand content increasing with depth.						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LOG OF DRILL HOLE

JOB NO.: SF05019 | LOGGED BY: A. Killeen & J. Seibold | DRILL HOLE NO.: B-9
PROJECT: Muni Power Plant | CHECKED BY: D. van Hoff | DRILLING DATE: 7/29/ & 8/01/2005
LOCATION: Lot between east ends of 25th St. & Cesar Chavez | ELEVATION: 12.5 feet
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash | DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
		3					"UPPER LAYERED SEDIMENTS (Quls)" CLAYEY SAND (SC), dark greenish gray, wet , loose, fine grained sand, approximately 40% medium plasticity silty clay fines.						
60		13		1.20			SANDY CLAY (CL), dark greenish gray, moist to wet, stiff, medium plasticity clay, approximately 40% fine grained sand.						
65		61					POORLY GRADED SAND with SILT (SP-SM), dark yellowish orange with trace dark gray mottling, moist, dense to very dense, fine grained sand.						
70		50/6"					Fine angular gravel lens.						GS
75		50/5.5"					SILTY SAND (SM), moderate to dark yellowish brown, wet, very dense, poorly graded fine grained sand, approximately 15-20% silty fines, trace angular chert gravel clasts to 1-inch.						
80		19					"OLDER BAY MUD (Qobm)" SILTY CLAY (CL), grayish blue green, moist, very stiff, medium plasticity silty clay, trace fine grained sand.						
85													
							FAT CLAY (CH), dark greenish gray, moist, medium stiff.						
90		12	0.54	1.10				69	51			1339	
95							Increasing sand content.						
100		50/5"					"UPPER LAYERED SEDIMENTS (Quls)" SILTY SAND (SM), olive gray to yellow orange, wet, very dense, fine grained sand, trace coarse gravel.						
105							1) Bottom boring at 100 feet and 11 inches. 2) Groundwater measured at 10.2 feet on 8/01/05. 3) Boring backfilled with cement grout.						

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: Rotary Wash, 5-inch dia.
LOGGED BY: A. Killeen
CHECKED BY: J. Seibold
DRILL HOLE NO.: B-10
DRILLING DATE: 7/25/2005
ELEVATION: 12.5 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							"ARTIFICIAL FILL (af)" POORLY GRADED GRAVEL (GP), light brown, dry, loose, underlain by drainage fabric at 6-inches depth.						
							CLAYEY GRAVEL (GC), light to dark brown, dry, loose, angular gravel to 1-inch, minor sand, red brick fragments.						
5		47					Dense, serpentinite fragments.	132	10				
							Brick and asphalt fragments.						
							LEAN CLAY (CL), dark brown to black, moist, abundant fragments, medium plasticity.						
10		21					At approximately 10 to 12 feet - abundant brick fragments and pieces in sandy clay matrix, minor granitic fragments.						
							POORLY GRADED SAND with CLAY (SP-SC), gray to black, wet, fine grained sand, abundant brick fragments.						
15		32					SILTY SAND with GRAVEL (SM), dark brown to black, wet, dense, angular serpentinite fragments, fragment and pieces of bricks, concrete, ceramic tile, cobblestones.						GS
20													
		19					Black, medium dense, metal at 22 feet, wood fragments, glass fragments, abundant organics, oily substance from 21 to 24 feet.						GS
25		13					SILTY GRAVEL with SAND (GM), black, medium dense, wood and glass fragments, abundant organics, oily substance from 24 to 26 feet.						GS
30		18					"YOUNGER BAY MUD (Qybm)" FAT CLAY (CH), dark greenish gray, moist, stiff, high plasticity clay, shell fragments up to 1-inch.						
35							1) Bottom boring at 31.5 feet. 2) Groundwater measured at 11.7 feet on 7/25/05. 3) Temporary 3-inch piezometer set to 30 feet in boring on 7/25/05 for geophysics testing. 4) Temporary piezometer destroyed on 8/02/05 by drilling out the top 5 feet and filling it with cement grout.						
40													
45													
50													

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash

LOGGED BY: D. van Hoff
CHECKED BY: A. Killeen

DRILL HOLE NO.: B-11
DRILLING DATE: 7/26/2005
ELEVATION: 14.0 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							"ARTIFICIAL FILL (af)" POORLY GRADED GRAVEL (GP), approximately ¾-inch crushed recycled concrete, overlaying black geotextile fabric. GRAVELLY CLAY (CL), dark grayish brown, wet, stiff. CLAYEY GRAVEL (GC), dark grayish brown, moist, medium dense, with chert and brick fragments, minor wood debris.						
5		47					At 3 feet - dark greenish gray, inclusions of red decomposed brick. Minor serpentinite clasts to 3-inches. GRAVELLY SILT (ML), olive brown, moist, medium dense. Approximately 6-inch thick layer of mottled brick red and brown Silty Gravel.						Corr
10		29					At 10 feet - dark grayish brown, strong petroleum odor. At 11 feet - black, metal debris, wire and marble tile fragments. At 12 feet - approximately 1½- foot thick concrete slab/block.						
15		27					POORLY GRADED GRAVEL with SILT and SAND (GP-GM), dark grayish brown, wet, medium dense, abundant brick and concrete debris, minor wood and glass fragments.	94	17	NV	NP		GS
20		5					Increased size and amount of brick fragments, decreasing silt content. At 22 to 25 feet - predominantly wood debris.						
25		55					CLAYEY SILT (ML), black, wet, stiff, abundant wood debris, slight petroleum odor. FILL DEBRIS, miscellaneous fill debris including, plastic, glass, brick in a clayey matrix.						
30		6					"YOUNGER BAY MUD (Qybm)" FAT CLAY (CH), dark gray, wet, medium stiff, minor shell fragments and silt.						
35		150 psi	0.42										
40		7											
45		50 psi	0.40					68	51	67	26		C UU
50		8	0.50				"UPPER LAYERED SEDIMENTS (Quls)"						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019 LOGGED BY: D. van Hoff DRILL HOLE NO.: B-11
PROJECT: Muni Power Plant CHECKED BY: A. Killeen DRILLING DATE: 7/26/2005
LOCATION: Lot between east ends of 25th St. & Cesar Chavez ELEVATION: 14.0 feet
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
		38					SILTY SAND (SM), dark greenish gray, wet, dense.	113	19				
60		35											GS
65		41											
70		22					Medium dense, minor local iron oxide staining.						
75							Gravelly layer. POORLY GRADED SAND (SP), dark yellowish brown, wet, very dense, fine to medium grained sand, trace silt.						
80		83											
85							"OLDER BAY MUD (Qobm)" CLAY (CL/CH), mottled greenish gray and yellow brown, wet, stiff.						
90		16						70	51			1547	
95													
100		81					"UPPER LAYERED SEDIMENTS (Quls)" SILTY SAND (SM), olive brown, wet, dense.						
105							1) Bottom boring at 101.5 feet. 2) Groundwater measured at 12.5 feet on 7/26/05. 3) Temporary 3-inch piezometer set in boring to 100 feet on 7/26/05 for geophysics testing. 4) Temporary piezometer destroyed on 8/02/05 by drilling out the top 5 feet and filling it with cement grout.						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash

LOGGED BY: J. Seibold
CHECKED BY: A. Killeen

DRILL HOLE NO.: B-12
DRILLING DATE: 7/26/2005
ELEVATION: 14.0 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
		50/5.5"					"ARTIFICIAL FILL (af)" SILTY GRAVEL (GM) with SAND, brownish gray, damp, dense, mixed gravel, fine to coarse subrounded to angular gravel clasts, minor brick fragments, matrix of sandy silt from decomposed serpentinite.						
5		43					SILTY SAND (SM) with GRAVEL, yellowish brown, damp, dense, fine grained sand, with approximately 25% gravel, minor brick fragments and serpentinite clasts.						
10		27					Increased clayey fines, medium dense, with brick, glass and metal fragments, decreased gravel to approximately 15%.						
15		50/5"					POORLY GRADED SAND with SILT and GRAVEL (SP-SM), olive gray to olive black, moist, very dense, mixed gravel, fine to coarse gravel - up to 1 1/2-inch, rounded to angular, with fine to medium grained sand, serpentinite derived gravel, minor brick fragments.						GS
20		30					Concrete fragments, black staining, moderate odor.						
25		16											
30		7	0.6				"YOUNGER BAY MUD (Qybm)" FAT CLAY (CH), dark greenish gray, moist, soft to medium stiff, with trace scattered shell fragments, slight H_2S odor.						
35		100 psi											
40		125 psi					Large clam shell in sampler shoe, no odor.	69	55	70	26		C UU
45		4	0.26	0.7			Becoming soft.						
50			0.26	0.65									
							"UPPER LAYERED SEDIMENTS (Quls)"						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash
LOGGED BY: J. Seibold
CHECKED BY: A. Killeen
DRILL HOLE NO.: B-12
DRILLING DATE: 7/26/2005
ELEVATION: 14.0 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
		50/5"					POORLY GRADED SAND (SP) with SILT, dark greenish gray, wet, dense, fine grained sand, approximately 5% non-plastic silty fines.						
60		16					SANDY CLAY (CL), greenish gray, moist, stiff, low to medium plastic clay with approximately 40% fine grained sand. Increased sand from 61.25 to 61.5 feet.						
							CLAYEY SAND (SC), grayish green grading to dusky yellow green, moist, dense, fine grained sand with approximately 30% low plasticity silty clay fines.						
65		49											Corr
70		9					SANDY CLAY (CL), pale to moderate yellowish brown with dark orange mottling/veining, moist, stiff, medium plasticity clay with 25% fine grained sand.						
75		50/6"					POORLY GRADED SAND with SILT (SP-SM), moderate yellowish brown, wet, very dense, fine grained sand.						GS
80		50/6"					Olive to dark yellowish brown.						
85							Clayey cuttings.						
							"OLDER BAY MUD (Qobm)" FAT CLAY (CH), grayish blue green with dark yellowish orange mottling, moist, stiff.						
90		16	0.67	1.2				67	57				
95													
							"UPPER LAYERED SEDIMENTS (Quls)" SILTY SAND (SM), dark greenish gray, moist, very dense, fine grained sand with approximately 35% clayey silt fines.						
100		90											
							1) Bottom boring at 101.5 feet. 2) Groundwater not measured. 3) Boring backfilled with cement grout.						
105													

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: Rotary Wash, 5-inch dia.

LOGGED BY: J. Seibold
CHECKED BY: A. Killeen

DRILL HOLE NO.: B-13
DRILLING DATE: 7/25/2005
ELEVATION: 14.0 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							"ARTIFICIAL FILL (af)"						
		50/6"					SILTY GRAVEL (GM) with SAND, brown and grayish green, damp, very dense, mixed well graded gravel up to 2 1/2-inch, rounded to angular, minor flat clasts, minor serpentinite derived fill, trace brick fragments. 3.5 to 5 feet: serpentinite cobbles and boulders in fill.						
5		21					GRAVELLY SAND TO SANDY GRAVEL (SW/GW), mixed colors of grayish green, red, orange, yellow, black, dry to damp, medium dense, gravel clasts composed of brick, serpentinite, yellow brick, up to 2-inch, angular to subangular, fine sand comprised of crushed brick and serpentinite, trace glass fragments, medium???? clayey blebs.						
10		17					10 to 11.5 feet: grayish blue (crushed cement?) with cobbles and olive gray clayey blebs.						
15		4					POORLY GRADED SAND with SILT and GRAVEL (SP-SM), gray grading to black (contamination staining), wet, loose, well graded subangular sand (coarse fraction), with wood fragments, brick fragments, cardboard, significant oily substance on wood and cardboard or chipboard from 16 to 16.5 feet. Significant oil in drilling mud.						GS
20		24					20 to 21.5 feet: medium dense with large wood fragment with oily staining.						
25		34					25 to 26.5 feet: dense with brick fragment, metal shavings, nail, gravel, black staining.						
30		6 / 10					"YOUNGER BAY MUD (Qybm)" FAT CLAY (CH), dark greenish gray, moist to wet, soft, with scattered shell fragments.						
35							1) Bottom boring at 33 feet. 2) Groundwater measured at 12.0 feet on 7/25/05. 3) Boring backfilled with cement grout.						
40													
45													
50													

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 8-inch Core Barrel to 16 feet, then 5-inch Rotary Wash

LOGGED BY: J. Seibold
CHECKED BY: A. Killeen

DRILL HOLE NO.: B-14
DRILLING DATE: 7/22 & 7/25/2005
ELEVATION: 12.5 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							"ARTIFICIAL FILL (af)" 1-foot SILTY GRAVEL (GM), over geotextile fabric.						
		34					SILTY GRAVEL (GM) with SAND, light yellowish brown, damp, sub angular to angular fine to coarse gravel up to 3-inches, predominantly franciscan derived clasts, angular fine to coarse grained sand, silt derived from decomposed serpentinite.						
5		17					CLAYEY SILT (ML) with GRAVEL, greenish gray and yellowish orange, damp, stiff, coarse gravel to 3-inches, predominantly Franciscan derived fill, trace to minor concrete, metal, and wood fragments.						
10		9					SANDY CLAY (CL) with SILT, mottled olive gray and grayish black, moist, stiff, Franciscan derived fines, low plasticity fines, local thin sand lenses, minor black staining.						
15							FILL DEBRIS, wet, miscellaneous fill debris including concrete rubble, wire, wood, brick, grave,l and cobbles. At 12 feet - cored though approximately 8-inch thick concrete slab/block.						
20													
25		4					SANDY GRAVEL (GW), wet, mixed gravel, fine subrounded to coarse subangular clasts, angular medium to coarse grained sand, abundant wood and brick fragments, minor glass fragments, trace concrete fragments.						
30							"YOUNGER BAY MUD (Qybm)" FAT CLAY (CH), dark greenish gray, moist, soft, trace shell fragments.						
35		50 psi		0.55			Scattered shell fragments.			65	27		C
40		3	0.33	0.55									
45		100 psi		0.65									
50		0 0 psi					Sample highly disturbed (slough?).						
							"UPPER LAYERED SEDIMENTS (Quls)"						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019 | LOGGED BY: J. Seibold | DRILL HOLE NO.: B-14
PROJECT: Muni Power Plant | CHECKED BY: A. Killeen | DRILLING DATE: 7/22 & 7/25/2005
LOCATION: Lot between east ends of 25th St. & Cesar Chavez | ELEVATION: 12.5 feet
DRILLING METHOD: 8-inch Core Barrel to 16 feet, then 5-inch Rotary Wash | DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
		14					SILTY SAND (SM), dark greenish gray, wet, fine grained sand. Thin (approximately 8-inches) Fat Clay seam/lens. At 56 feet - medium dense, local decreased silt content to approximately 5%.						
60		25											
65		39					Dense.						GS
70		32					CLAYEY SAND (SC), dark yellowish orange, moist, dense, fine grained sand, approximately 35% low plasticity clay fines.						
75		50/5"					POORLY GRADED SAND (SP), dusky yellow to light olive gray, wet, very dense, fine grained sand.						
80		57					SILTY SAND (SM), dusky yellow, wet, very dense, fine grained sand, silt content ranges from approximately 10 to 20%.						GS
85													
							"OLDER BAY MUD (Qobm)" SANDY CLAY (CL), grayish blue green, moist.						
90		24	0.9	2.2			FAT CLAY (CH), grayish blue green with dark yellowish orange mottling, moist, stiff.						
95													
100		10		0.6			"UPPER LAYERED SEDIMENTS (Quls)" CLAYEY SAND (SC), dark greenish gray, moist, dense, fine grained sand, approximately 30% high plasticity clayey fines.						
105							1) Bottom boring at 101.5 feet. 2) Groundwater measured at 11.7 feet on 7/22/05. 3) Boring backfilled with cement grout.						

LOG_DRILL_HOLE SF05019.GPJ GTC.GDT 9/14/05

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash

LOGGED BY: J. Seibold
CHECKED BY: A. Killeen

DRILL HOLE NO.: B-15
DRILLING DATE: 7/20 & 7/21/2005
ELEVATION: 14.5 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							"ARTIFICIAL FILL (af)" CLAYEY GRAVEL (GC) with SAND, dark reddish brown, damp, very dense, fine subrounded to coarse angular gravel clasts, coarse gravel predominantly chert and greenstone clasts, angular coarse grained sand, low plasticity clay matrix.						
		50/3"											
5		50/2"					CLAYEY SAND with GRAVEL (SC), light yellowish brown to dark gray, moist, very dense, fine to coarse grained sand, coarse angular gravel, gravel comprised mostly of greenstone and serpentinite clasts, trace black staining.						
10		21					Wood fragments. Medium dense, local sandstone cobbles.	107	19	35	16		GS
							Scattered wood and brick fragments.						
15		30					Abundant brick and wood fragments.						
20													
25													
30		100 psi					"YOUNGER BAY MUD (Qybm)" FAT CLAY (CH), dark greenish gray (5G 4/1), moist, medium stiff, scattered shell fragments.			72	27		C
								66	56				UU
35		4					Trace shells, increased silt content.						
40		75 psi											
				0.65									
45			0.38	0.55									
50													
				0.5									

LOG OF DRILL HOLE

JOB NO.: SF05019 | LOGGED BY: J. Seibold | DRILL HOLE NO.: B-15
PROJECT: Muni Power Plant | CHECKED BY: A. Killeen | DRILLING DATE: 7/20 & 7/21/2005
LOCATION: Lot between east ends of 25th St. & Cesar Chavez | ELEVATION: 14.5 feet
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash | DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
		22					"UPPER LAYERED SEDIMENTS (Quls)" SILTY SAND (SM), grayish green, moist, dense, fine grained sand.						
60		80					Increasing clayey fines to borderline Silty Sand/Clayey Sand, very dense.	112	21				
65		64					Dark yellowish orange (10YR 6/6), decreased clayey fines to Silty Sand, low plasticity clayey silt fines						
70		39					Grading between dusky yellow and yellowish orange.						
							Minor coarse sand and fine subrounded gravel.						
							Reddish brown silty clay seam/lens.						
75		50/6"											
80		50/6"					Moderate yellowish brown, decreased fines, becoming borderline Poorly Graded Sand/Silty Sand.						
85													
							LEAN TO FAT CLAY (CL/CH), yellowish gray, moist, medium to high plasticity clay.						
							"OLDER BAY MUD (Qobm)" FAT CLAY (CH) with SILT, grayish blue green (5BG 5/2) to dark greenish gray (5G 4/1), moist, very stiff.						
90		35	>1.0 0.55	2.3 1.6				96	28			3722	
95													
100		23	0.35	1.2 1.6			Dark greenish gray.						
105							CLAYEY SAND/SANDY CLAY (SC/CL), dark greenish gray, moist.						
							"UPPER LAYERED SEDIMENTS (Quls)" SILTY SAND (SM), dusky yellow, moist, very dense, fine grained sand, approximately 5% low plasticity clayey silt fines.						

LOG OF DRILL HOLE

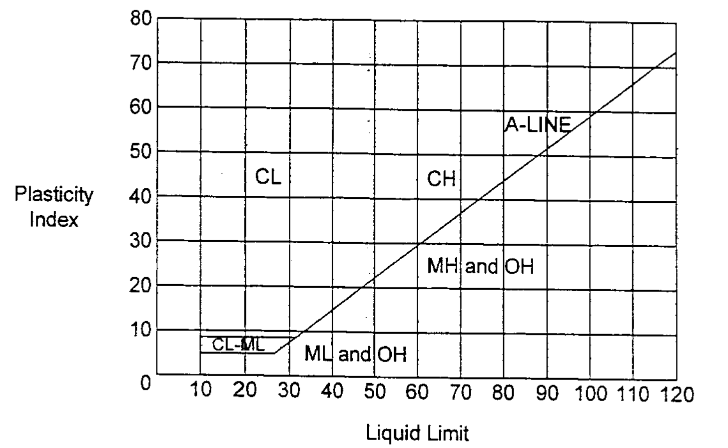
JOB NO.: SF05019
PROJECT: Muni Power Plant
LOCATION: Lot between east ends of 25th St. & Cesar Chavez
DRILLING METHOD: 8-inch Core Barrel and 5-inch Rotary Wash

LOGGED BY: J. Seibold
CHECKED BY: A. Killeen

DRILL HOLE NO.: B-15
DRILLING DATE: 7/20 & 7/21/2005
ELEVATION: 14.5 feet
DATUM: NAVD 1988

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (TSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
		50/6"											GS
115													
120		50/5"					Light olive gray, decreased fines.						
125													
							"OLDER BAY MUD (Qobm)" FAT CLAY (CH),dark greenish gray, moist, stiff, high plasticity clay, approximately 5% silt.						
130		13		1.1									
135													
140		10		1.4									
145													
150							1) Bottom boring at 150 feet. 2) Groundwater measured at 12.7 feet on 7/20/05 and at 12.6 on 7/21/05. 3) Boring backfilled with cement grout and bentonite.						
155													
160													

PLASTICITY CHART - Used for Classification of Fine Grained Soils



BLOW COUNT - The number of blows required to drive the sampler the last 12 inches of an 18-inch drive. When the sampler is not advanced the last 12 inches, i.e. 100 blows in 9 inches, the notation is 100/9. Symbols designating various hammer weights, drop heights, and sampling methods are shown below. A number not enclosed by one of the following symbols indicates a Standard Penetration Test (SPT) using a 140-pound hammer and 30-inch drop height.

No. of blows	Driving Weight (pounds)	Drop Height (inches)	Sampling Method
( )	______	______	______
[ ]	______	______	______
{ }	______	______	______
< >	______	______	______

ADDITIONAL TESTS -

C:	Consolidation	GS:	Grain Size Distribution	SP:	Specific Gravity
CL:	Chloride	pH:	Hydrocarbon Ion Concentration	SU:	Sulphate
CORR:	Corrosion	PM:	Permeability	TD:	Triaxial Compression, Drained
CP:	Compaction	R:	R-Value	TDy:	Triaxial Compression, Dynamic
DS:	Direct Shear	RS:	Resistivity	TU:	Triaxial Compression, Undrained
EL:	Elasticity Index	S:	Swell	TRPH:	Total Recoverable Petroleum Hydrocarbons
EX:	Expansion	SE:	Sand Equivalent		

UNIFIED SOIL CLASSIFICATION SYSTEM

	MAJOR DIVISION		GROUP SYMBOL	DESCRIPTION	GRAPHIC LOG
COARSE GRAINED SOILS Over 50% By Weight Coarser Than No.200 Sieve Size	GRAVELLY SOILS OVER 50% OF COARSE FRACTION LARGER THAN NO.4 SIEVE SIZE	CLEAN GRAVELLY SOILS LITTLE OR NO FINES	GW	well graded gravels or gravel-sand mixtures	
			GP	poorly graded gravels or gravel-sand mixtures	
		GRAVELLY SOILS WITH FINES OVER 12% FINES	GM	silty gravels or gravel-sand-silt mixtures	
			GC	clayey gravels or gravel-sand-clay mixtures	
	SANDY SOILS OVER 50% OF COARSE FRACTION SMALLER THAN NO.4 SIEVE SIZE	CLEAN SANDY SOILS LITTLE OR NO FINES	SW	well graded sands or gravelly sands	
			SP	poorly graded sands or gravelly sands	
		SANDY SOILS WITH FINES OVER 12% FINES	SM	silty sands or sand-silt mixtures	
			SC	clayey sands or sand-clay mixtures	
FINE GRAINED SOILS Over 50% By Weight Finer Than No.200 Sieve Size	SILTY AND CLAYEY SOILS LIQUID LIMIT LESS THAN 50		ML	inorganic silts, very fine sands, silty fine sands, clayey silts with slight plasticity	
			CL	inorganic clays, gravelly, sandy, silty, or lean clays, of low to medium plasticity	
			OL	organic clays or organic silts of low plasticity	
	SILTY AND CLAYEY SOILS LIQUID LIMIT GREATER THAN 50		MH	inorgainc silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
			CH	inorganic clays of high plasticity, fat clays	
			OH	organic clays or organic silts of medium to high plasticity	
	HIGHLY ORGANIC SOILS		Pt	peat or other highly organic soil, organic content greater than 60%	
				trash fill-landfill refuse (not a part of unified soil classification system)	

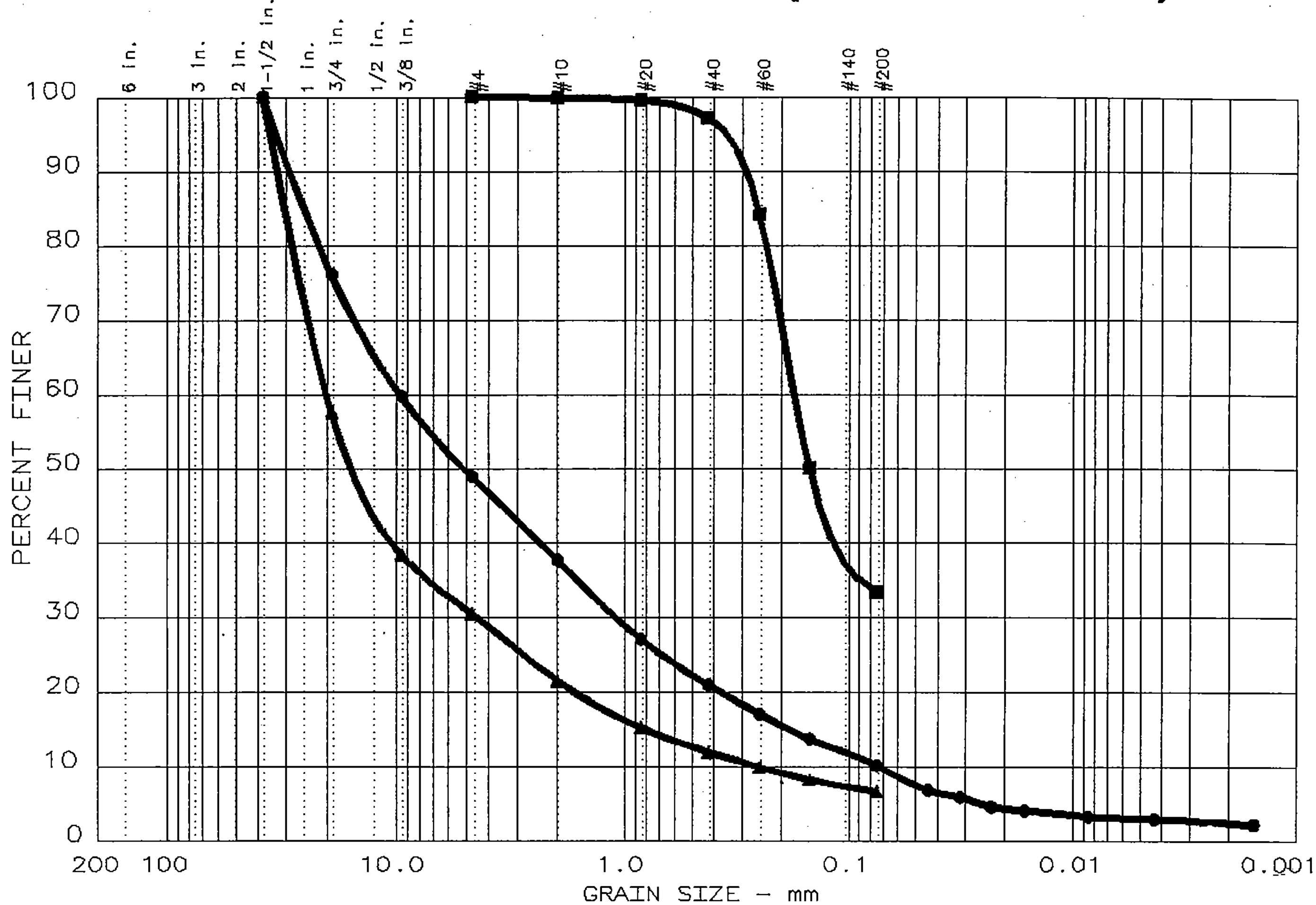
SAMPLE TYPES:
- UNDISTURBED SLEEVE
- DISTURBED
- UNSUCCESSFUL ATTEMPT
- STANDARD PENETRATION
- STANDARD PENETRATION NO RECOVERY
- SOIL CORE

CAVING:
- LIGHT CAVING
- HEAVY CAVING

WATER LEVEL:

PLATE A - 2
LEGEND TO LOGS

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	51.1	38.7	7.8	2.4	GW-GM		
▲	0.0	69.5	23.8	6.7		GP-GC		
■	0.0	0.0	66.6	33.4		SC		

SIEVE inches size	PERCENT FINER ●	▲	■
1.5	100.0	100.0	
0.75	76.1	57.5	
0.375	59.7	38.4	
	GRAIN SIZE		
D_{60}	9.66	20.07	0.17
D_{30}	1.10	4.49	
D_{10}	0.07	0.25	
	COEFFICIENTS		
C_c	1.72	3.98	
C_u	133.4	79.5	

SIEVE number size	PERCENT FINER ●	▲	■
4	48.9	30.5	100.0
10	37.6	21.5	99.8
20	27.1	15.2	99.5
40	21.0	12.0	97.1
60	17.1	10.0	84.2
100	13.7	8.3	50.2
200	10.2	6.7	33.4

Sample information:

● B-1 18-18.5'
Dk.gr.silty f-c GRAVEL w/sand.

▲ B-1 24-25.5'
Dk.gray clayey GRAVEL w/sand.

■ B-1 64-65.5'
Dk.gray clayey f-SAND.

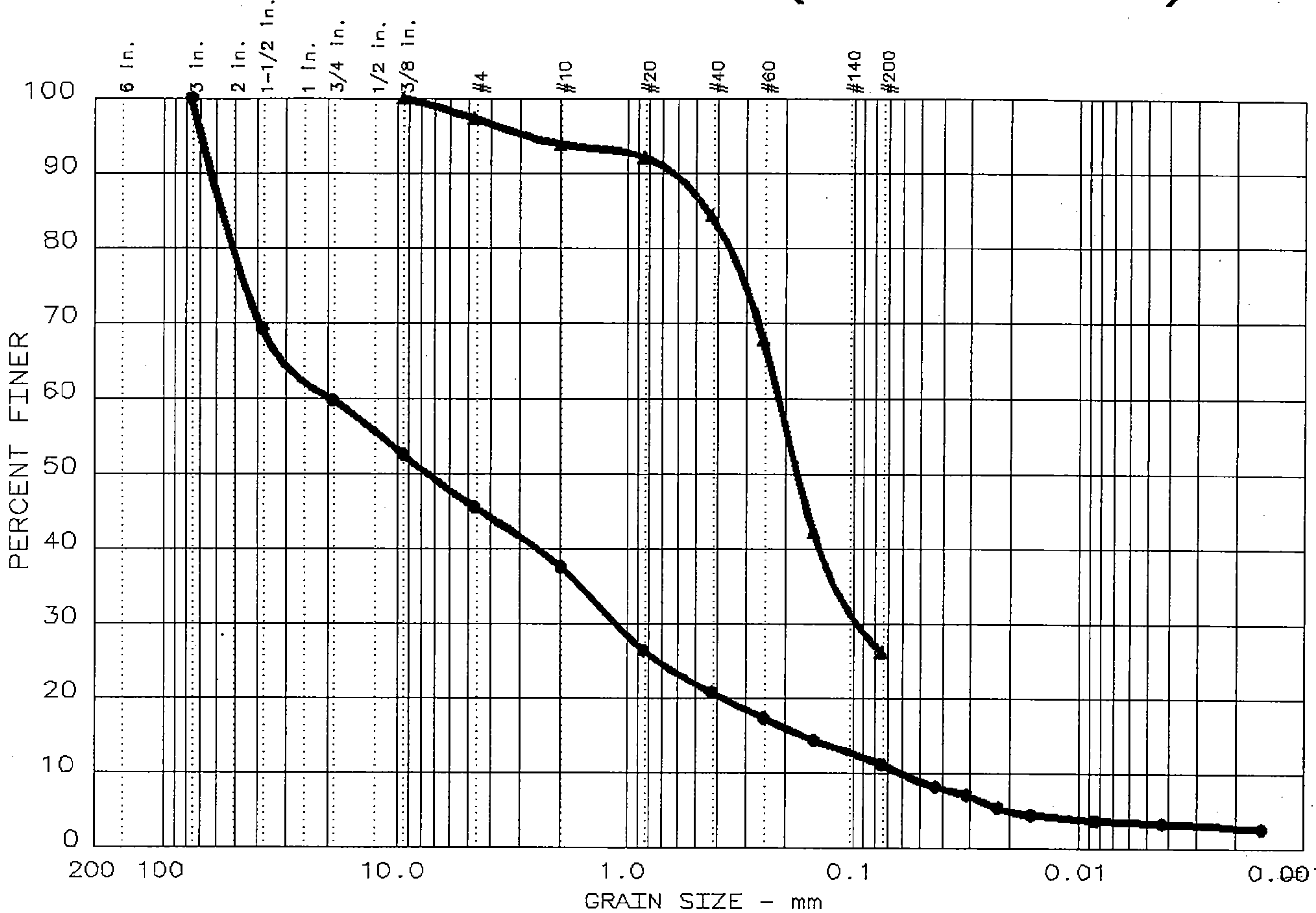
Remarks:

Soil Mechanics Lab

Project No.: SF05019
Project: MUNI Power Plant

Date: 8-30-05 Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	54.4	34.3	8.4	2.9	GW-GM		
▲	0.0	2.7	71.0	26.3		SM		

SIEVE inches size	PERCENT FINER ●	▲	
3	100.0		
1.5	69.2		
0.75	59.8		
0.375	52.5	100.0	
	GRAIN SIZE		
D_{60}	19.45	0.21	
D_{30}	1.13	0.10	
D_{10}	0.05		
	COEFFICIENTS		
C_c	1.11		
C_u	327.3		

SIEVE number size	PERCENT FINER ●	▲	
4	45.6	97.3	
10	37.5	93.8	
20	26.4	92.1	
40	20.8	84.3	
60	17.3	67.8	
100	14.4	42.3	
200	11.2	26.3	

Sample information:

● B-2 18-18.5'
Dk.gray m-c GRAVEL w/sand.

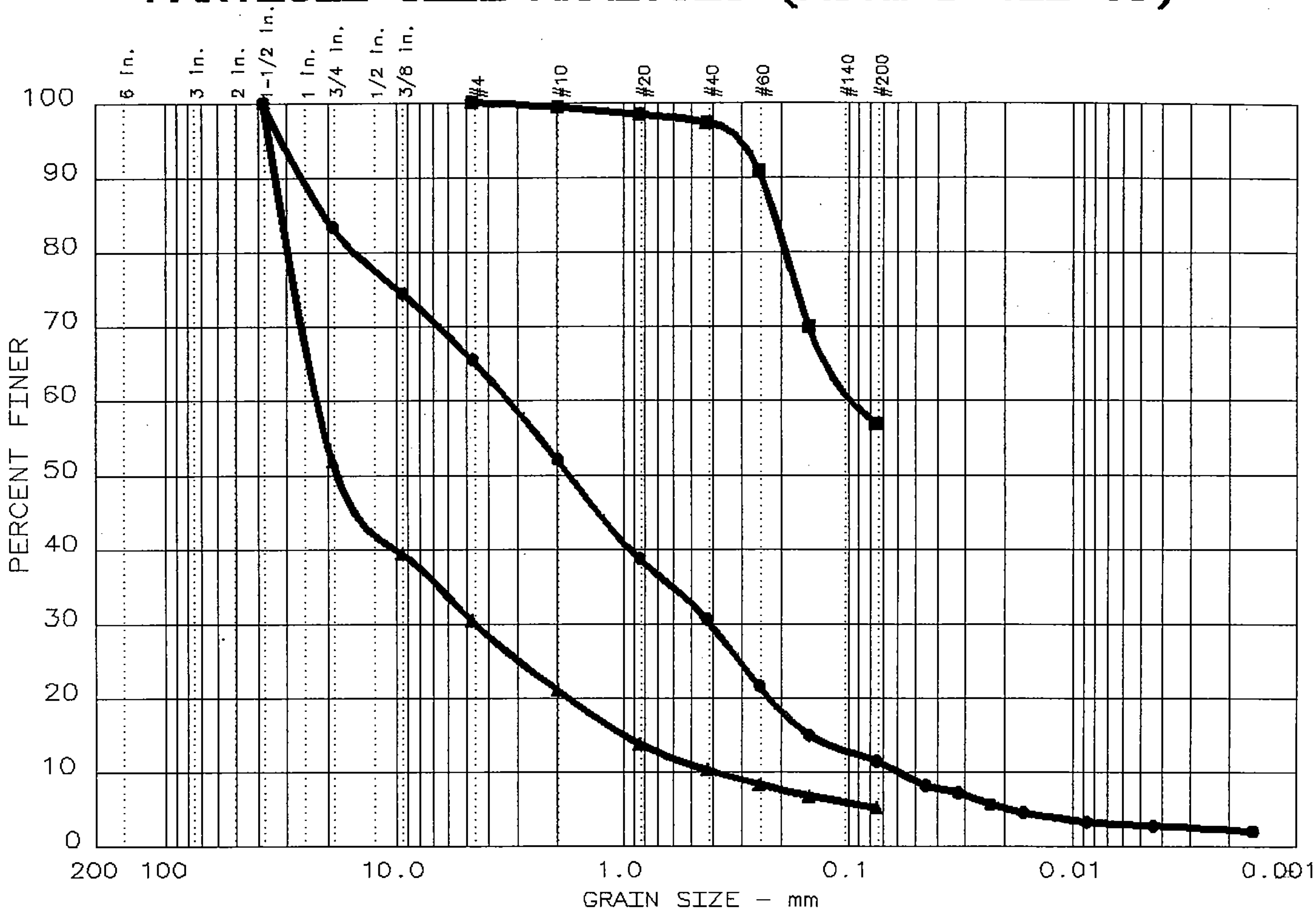
▲ B-2 65-66.5'
Olive brown silty SAND. Trace clay fines.

Remarks:

Soil Mechanics Lab

Project No.: SF05019
Project: MUNI Power Plant
Date: 8-30-05
Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	34.5	54.0	9.2	2.3	SP-SM		
▲	0.0	69.6	25.1	5.3		GW-GM		
■	0.0	0.0	43.2	56.8		CL		

SIEVE inches size	PERCENT FINER ●	▲	■
1.5	100.0	100.0	
0.75	83.4	51.9	
0.375	74.4	39.4	
GRAIN SIZE			
D_{60}	3.27	22.31	0.10
D_{30}	0.41	4.56	
D_{10}	0.06	0.39	
COEFFICIENTS			
C_c	0.85	2.37	
C_u	55.0	56.9	

SIEVE number size	PERCENT FINER ●	▲	■
4	65.5	30.4	100.0
10	52.1	21.0	99.4
20	38.8	13.8	98.4
40	30.6	10.3	97.3
60	21.7	8.3	90.9
100	15.0	6.8	70.0
200	11.5	5.3	56.8

Sample information:

● B-4 10-11.5'
Dk.brn. gravelly SAND.
Trace silt fines.

▲ B-4 24-25.5'
Dk.gray sandy GRAVEL.
Trace silt fines.

■ B-4 64-65.5'
Dk.gray sandy CLAY.

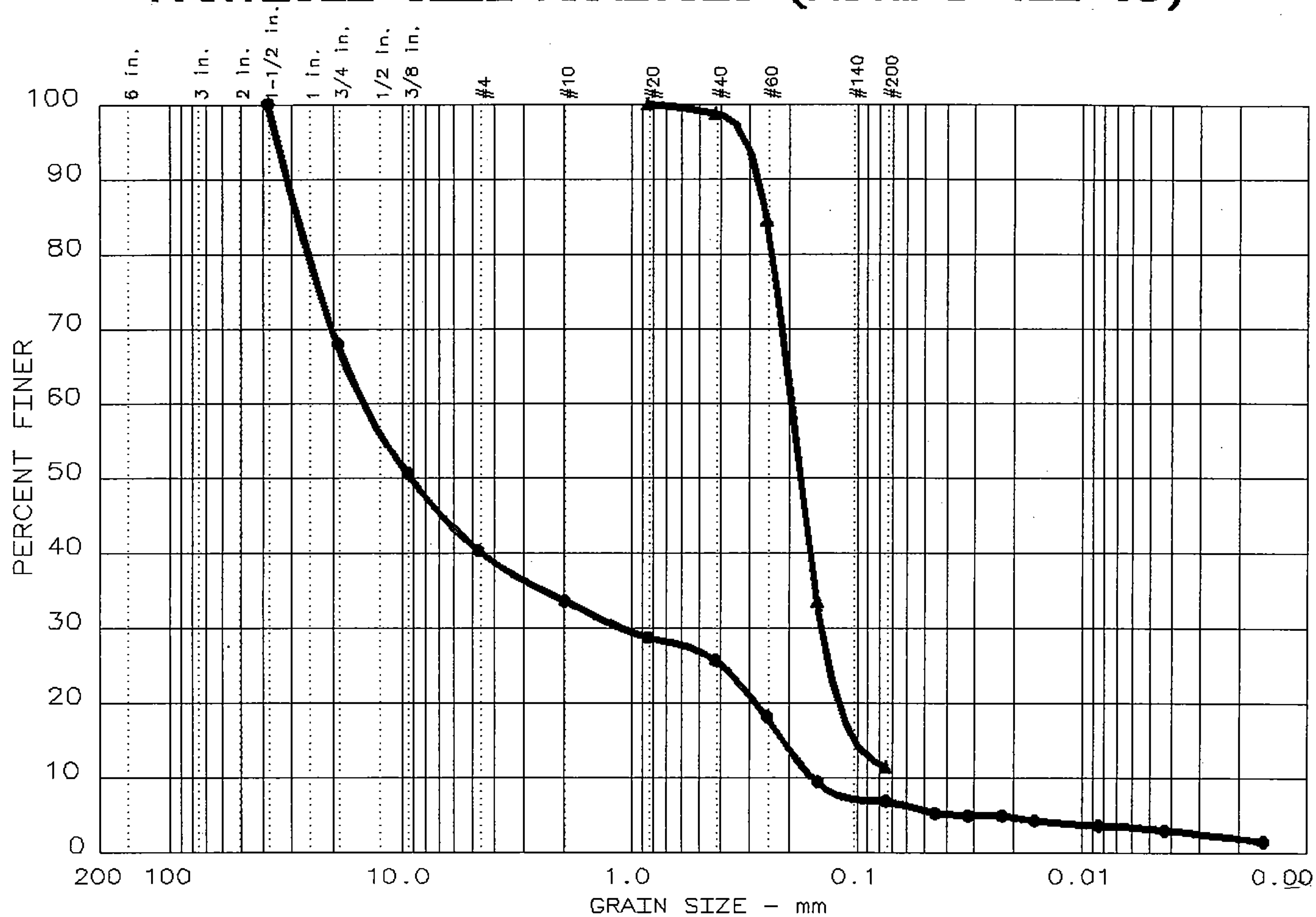
Remarks:

Soil Mechanics Lab

Project No.: SF05019
Project: MUNI Power Plant

Date: 8-30-05 Data Sheet No. ——

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	59.7	33.4	5.0	1.9	GP-GM		
▲	0.0	0.0	88.5	11.5		SP-SM		

SIEVE inches size	PERCENT FINER ●	▲	
1.5	100.0		
0.75	68.0		
0.375	50.6		
	GRAIN SIZE		
D_{60}	14.62	0.19	
D_{30}	1.12	0.14	
D_{10}	0.15		
	COEFFICIENTS		
C_c	0.55		
C_u	93.3		

SIEVE number size	PERCENT FINER ●	▲	
4	40.3		
10	33.6		
20	28.7	100.0	
40	25.7	98.7	
60	18.1	84.6	
100	9.4	33.6	
200	6.9	11.5	

Sample information:

● B-5 10-11.5'
Redish brn & gray sandy GRAVEL. Trace silt fines

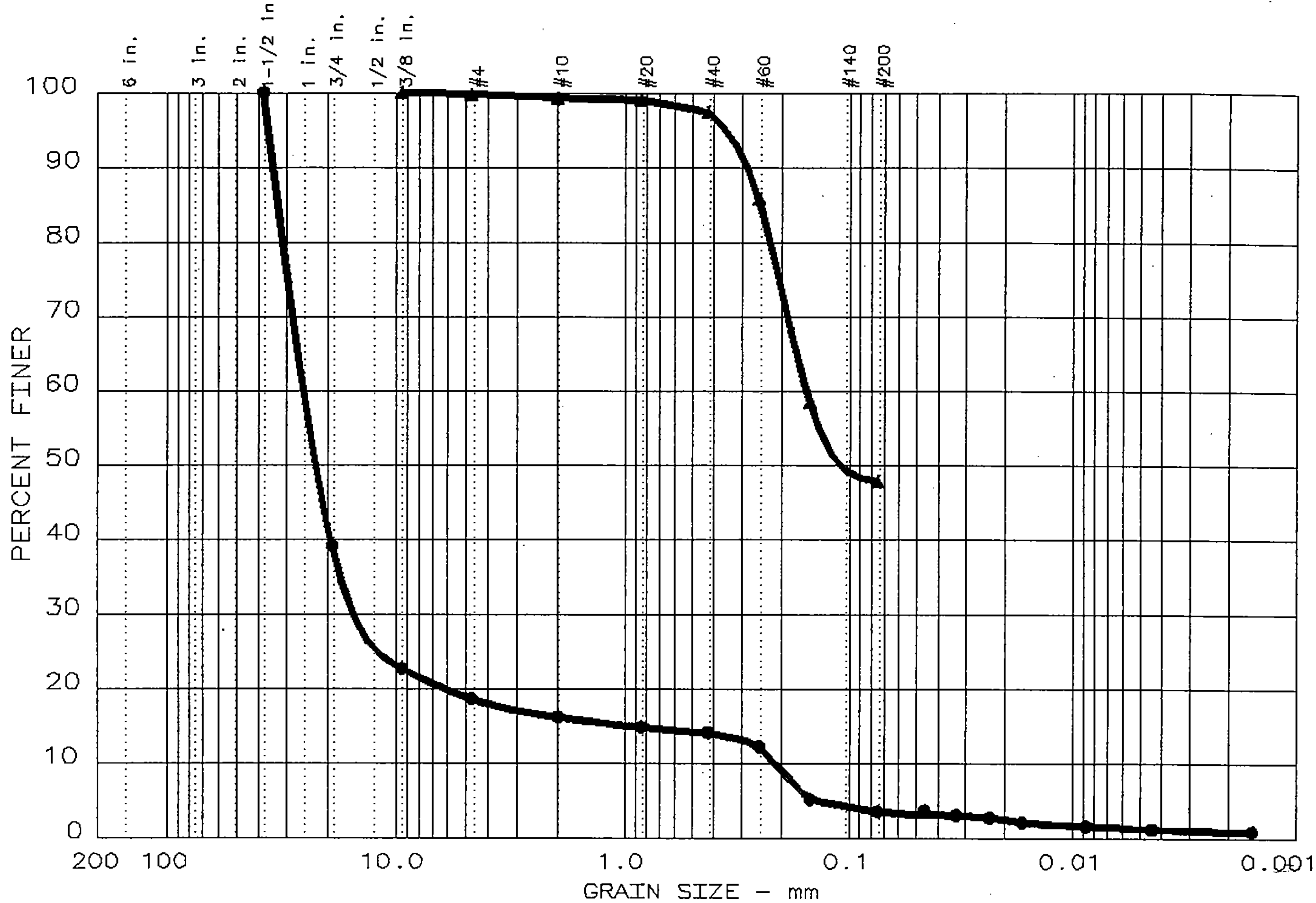
▲ B-5 70-71.5'
Dark brown fine SAND. Some silt fines.

Remarks:

Soil Mechanics Lab

Project No.: SF05019
Project: MUNI Power Plant
Date: 8-30-05
Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	81.4	15.1	2.6	0.9	GP		
▲	0.0	0.2	52.0	47.8		SC		

SIEVE inches size	PERCENT FINER ●	▲	
1.5	100.0		
0.75	39.1		
0.375	22.7	100.0	
	GRAIN SIZE		
D_{60}	25.35	0.15	
D_{30}	15.35		
D_{10}	0.21		
	COEFFICIENTS		
C_c	43.85		
C_u	119.7		

SIEVE number size	PERCENT FINER ●	▲	
4	18.6	99.8	
10	16.2	99.3	
20	14.8	99.0	
40	14.2	97.4	
60	12.3	85.8	
100	5.2	58.5	
200	3.5	47.8	

Sample information:

● B-6 19-20.5'
Black, solidified coarse TAR w/sand&silt(Trash)

▲ B-6 69-70.5'
Dark gray clayey SAND.

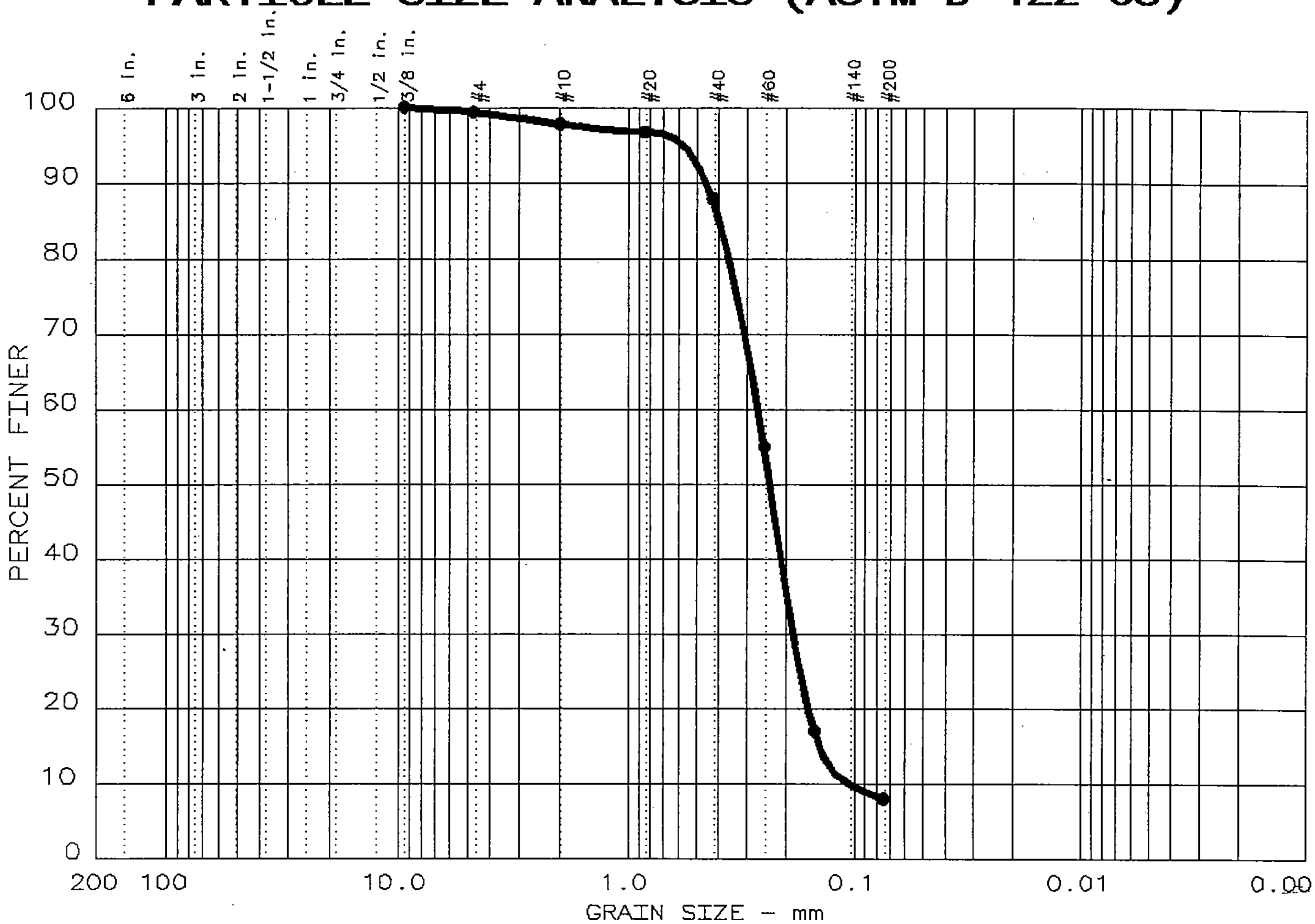
Remarks:

Soil Mechanics Lab

Project No.: SF05019
Project: Muni Power Plant

Date: 8-30-05 Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	0.7	91.3	8.0		SP-SM		

SIEVE inches size	PERCENT FINER ●		
0.375	100.0		
	GRAIN SIZE		
D_{60}	0.27		
D_{30}	0.18		
D_{10}	0.10		
	COEFFICIENTS		
C_c	1.22		
C_u	2.5		

SIEVE number size	PERCENT FINER ●		
4	99.3		
10	97.8		
20	96.7		
40	87.9		
60	55.0		
100	17.0		
200	8.0		

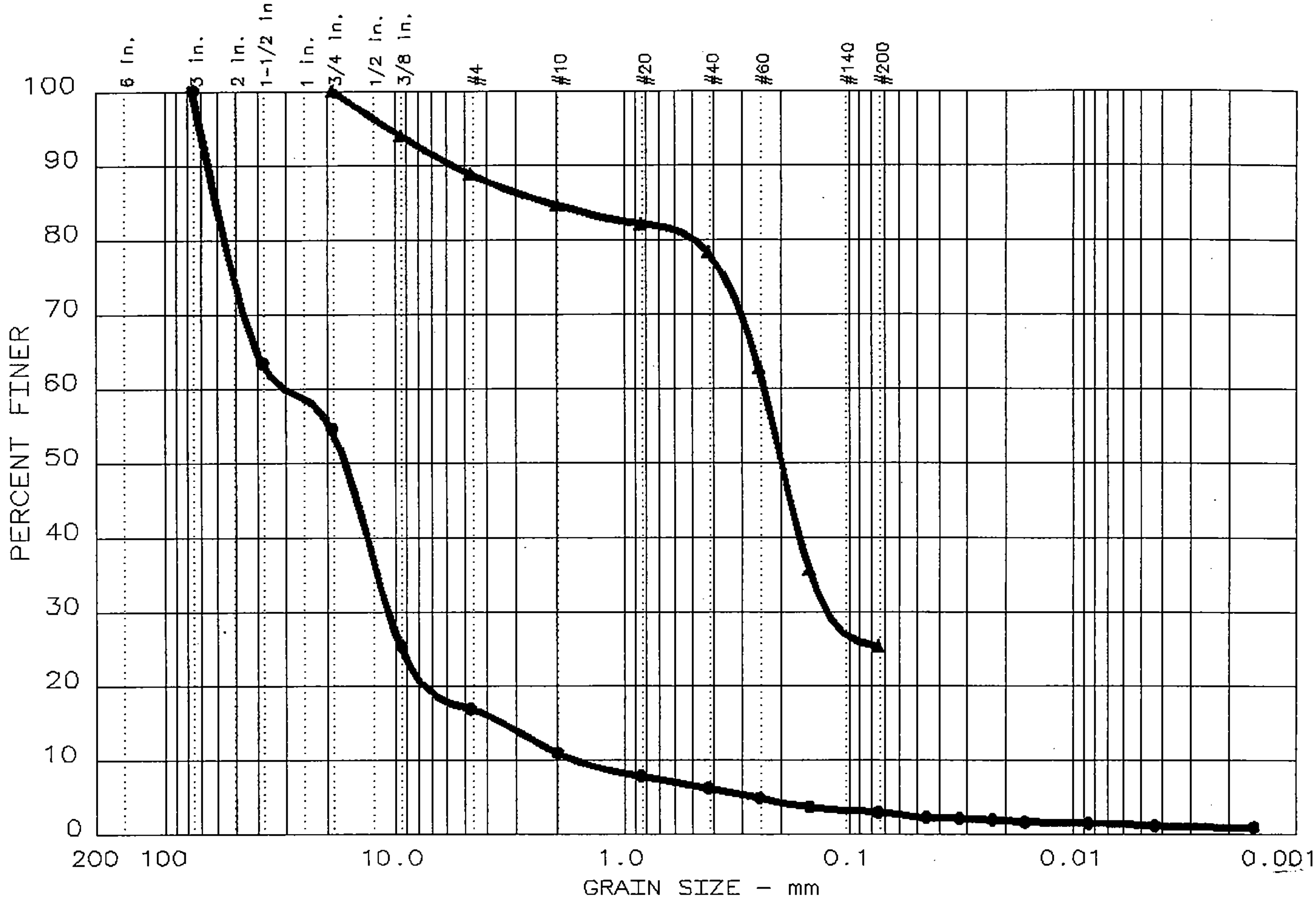
Sample information:
● B-7 75-76.5'
Bluish gray w/brown fine SAND.Trace silt.

Remarks:

Soil Mechanics Lab

Project No.: SF05019
Project: MUNI Power Plant
Date: 8-30-05
Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	83.1	14.0	2.0	0.9	GW		
▲	0.0	11.3	63.4	25.3		SM		

SIEVE inches size	PERCENT FINER ●	PERCENT FINER ▲	
3	100.0		
1.5	63.4		
0.75	54.6	100.0	
0.375	25.4	93.9	
	GRAIN SIZE		
D_{60}	30.90	0.24	
D_{30}	10.72	0.12	
D_{10}	1.67		
	COEFFICIENTS		
C_c	2.21		
C_u	18.4		

SIEVE number size	PERCENT FINER ●	PERCENT FINER ▲	
4	16.9	88.7	
10	10.9	84.6	
20	7.8	82.1	
40	6.2	78.3	
60	4.9	62.7	
100	3.7	35.7	
200	2.9	25.3	

Sample information:

● B-8 20-21.5'
Gray/brick-red m-c GRAVEL.

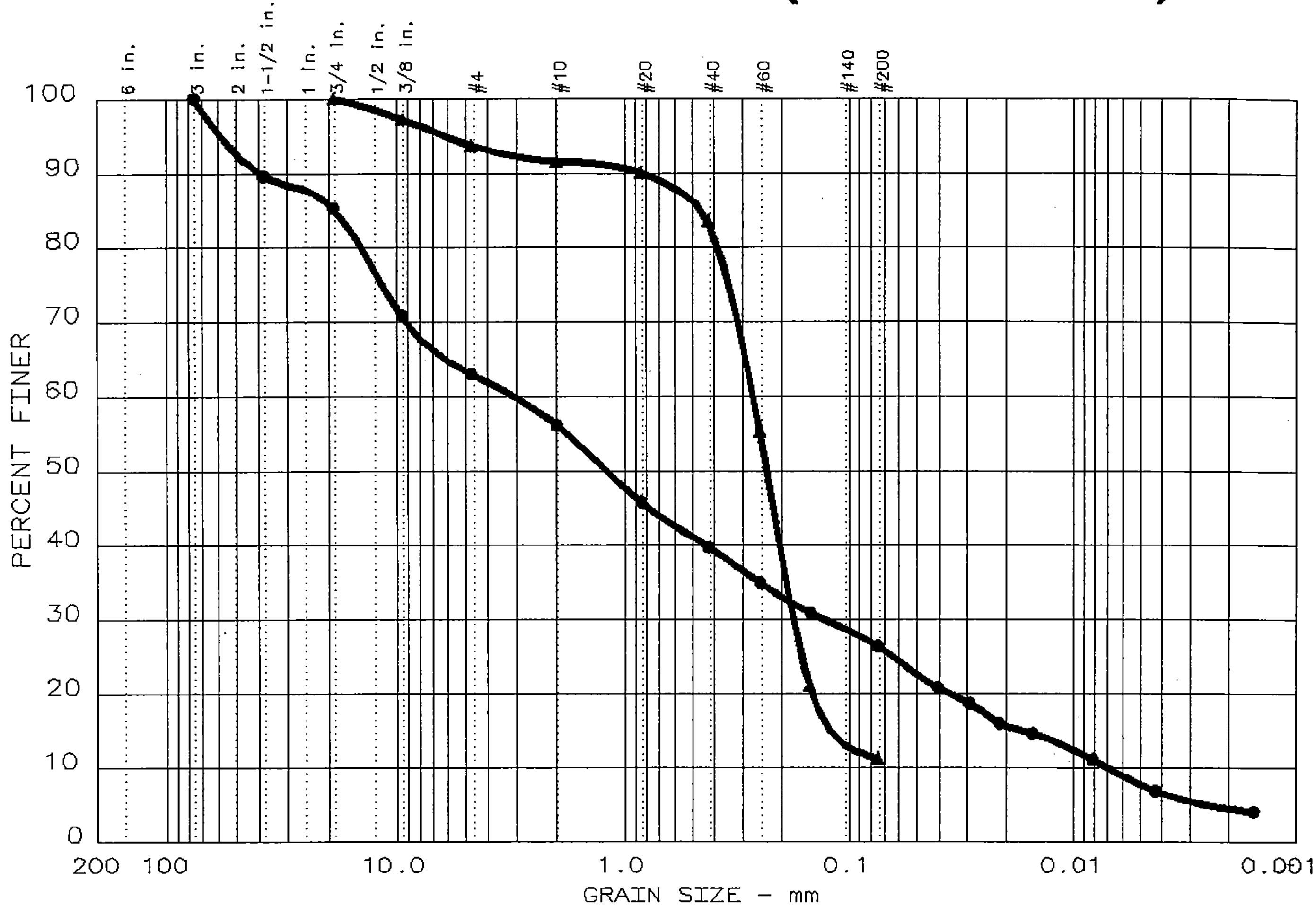
▲ B-8 70-71.5'
Dk.gray silty f-SAND. Trace gravel.

Remarks:

Soil Mechanics Lab

Project No.: SF05019
Project: Muni Power Plant
Date: 8-30-05
Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	37.0	36.6	21.8	4.6	GM		
▲	0.0	6.3	82.4	11.3		SP-SM		

SIEVE inches size	PERCENT FINER ●	▲	
3	100.0		
1.5	89.6		
0.75	85.2	100.0	
0.375	70.7	97.2	
	GRAIN SIZE		
D_{60}	3.09	0.27	
D_{30}	0.13	0.18	
D_{10}	0.00		
	COEFFICIENTS		
C_c	0.79		
C_u	441.6		

SIEVE number size	PERCENT FINER ●	▲	
4	63.0	93.7	
10	56.1	91.6	
20	45.8	90.0	
40	39.7	83.5	
60	34.9	55.3	
100	30.9	21.1	
200	26.4	11.3	

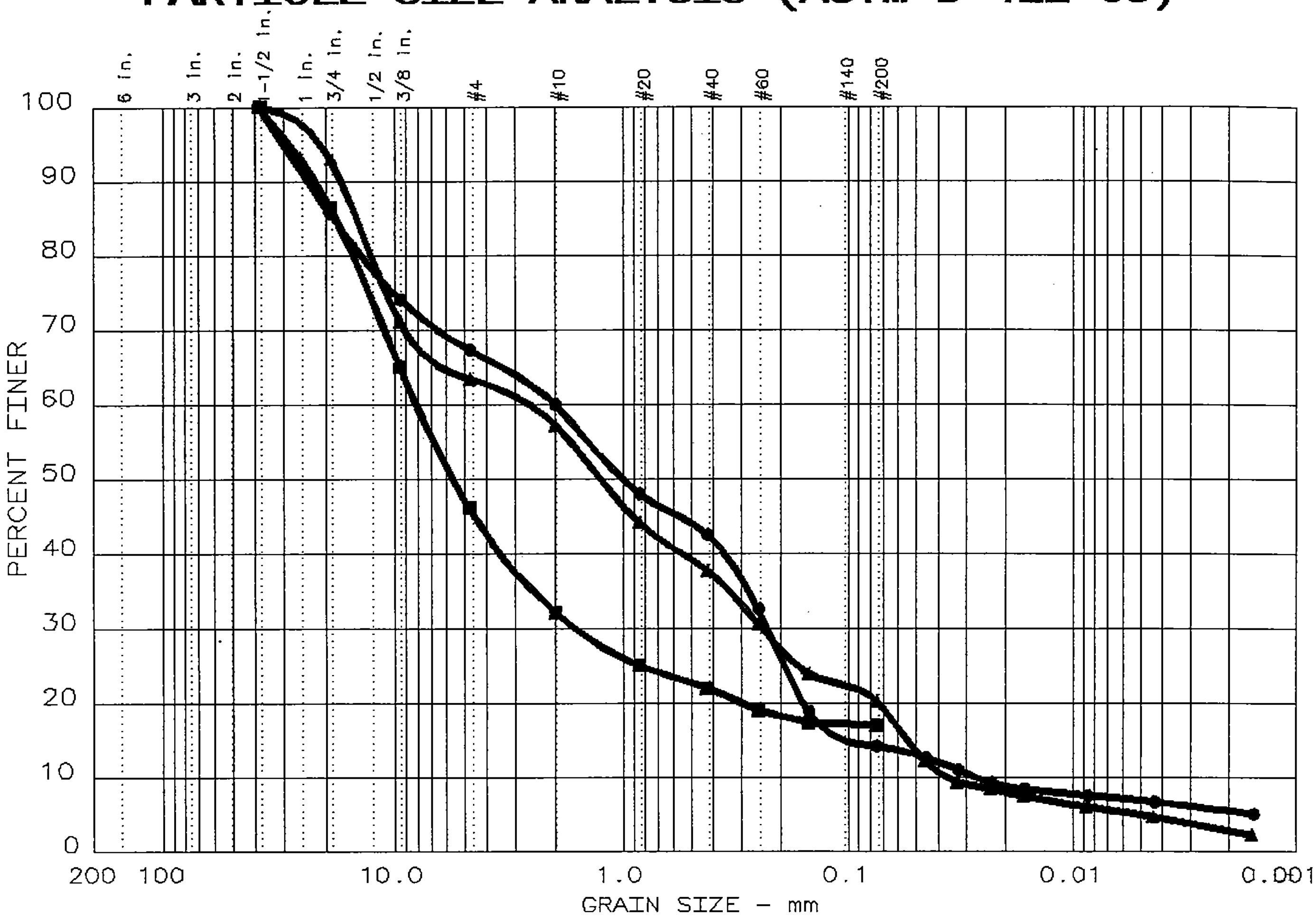
Sample information:
● B-9 12-13'
Gray/black f-c GRAVEL w/glass pcs.& org.trash
▲ B-9 70-71.5'
Dk. brown fine SAND. Some silt fines.

Remarks:

Soil Mechanics Lab

Project No.: SF05019
Project: MUNI Power Plant
Date: 8-30-05
Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	32.7	53.1	8.7	5.5	SM		
▲	0.0	36.6	43.1	17.4	2.9	SM		
■	0.0	53.9	29.0	17.1		GM		

SIEVE inches size	PERCENT FINER ●	▲	■
1.5	100.0	100.0	100.0
0.75	85.6	92.9	86.5
0.375	74.1	71.1	65.0
	GRAIN SIZE		
D_{60}	2.00	2.60	8.04
D_{30}	0.23	0.24	1.63
D_{10}	0.02	0.03	
	COEFFICIENTS		
C_c	0.96	0.60	
C_u	73.6	71.6	

SIEVE number size	PERCENT FINER ●	▲	■
4	67.3	63.4	46.1
10	59.9	57.2	32.1
20	47.9	44.2	25.0
40	42.5	37.6	21.9
60	32.5	30.7	19.1
100	18.9	23.8	17.4
200	14.2	20.3	17.1

Sample information:

● B-10 14-15.5'
Dk.gray gravelly SAND.
Trace silt & clay fines

▲ B-10 21-22.5'
Grayish black silty f-c
SAND w/gravel.

■ B-10 24-25.5'
V.dk.brn. to black f-c
GRAVEL w/glass & trash.

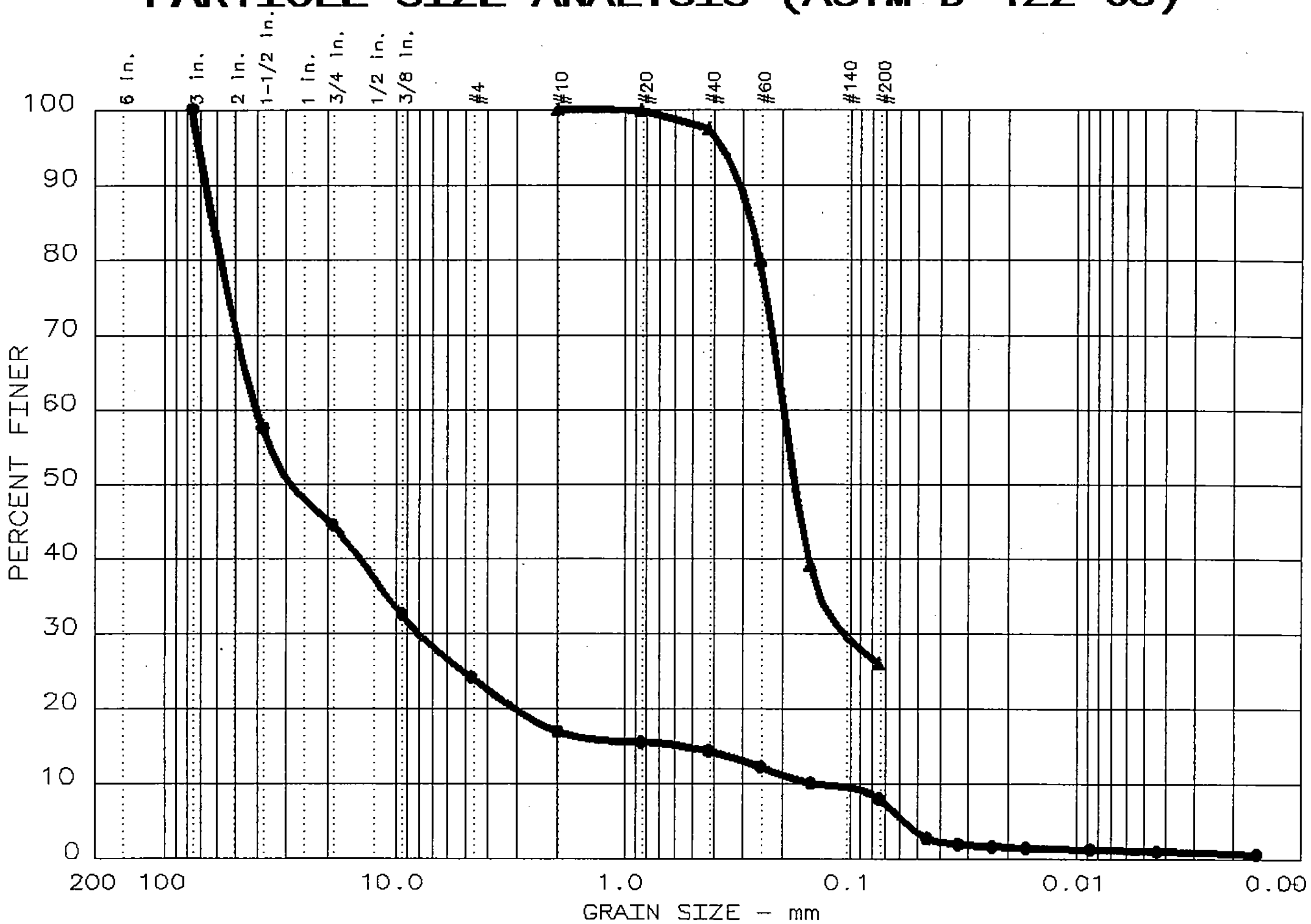
Remarks:

Soil Mechanics Lab

Project No.: SF05019
Project: MUNI Power Plant

Date: 8-30-05 Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	75.8	16.1	7.3	0.8	GP-GM		
▲	0.0	0.0	74.0	26.0		SM		

SIEVE inches size	PERCENT FINER ●	▲	
3	100.0		
1.5	57.6		
0.75	44.6		
0.375	32.6		
	GRAIN SIZE		
D_{60}	40.27	0.20	
D_{30}	7.94	0.11	
D_{10}	0.13		
	COEFFICIENTS		
C_c	11.75		
C_u	302.0		

SIEVE number size	PERCENT FINER ●	▲	
4	24.2		
10	16.9	100.0	
20	15.6	99.9	
40	14.4	97.4	
60	12.2	80.0	
100	10.1	39.2	
200	8.0	26.0	

Sample information:

● B-11 16-16.5'
Dk.gr.,red/brn. GRAVEL, BRICK & glass w/sand.

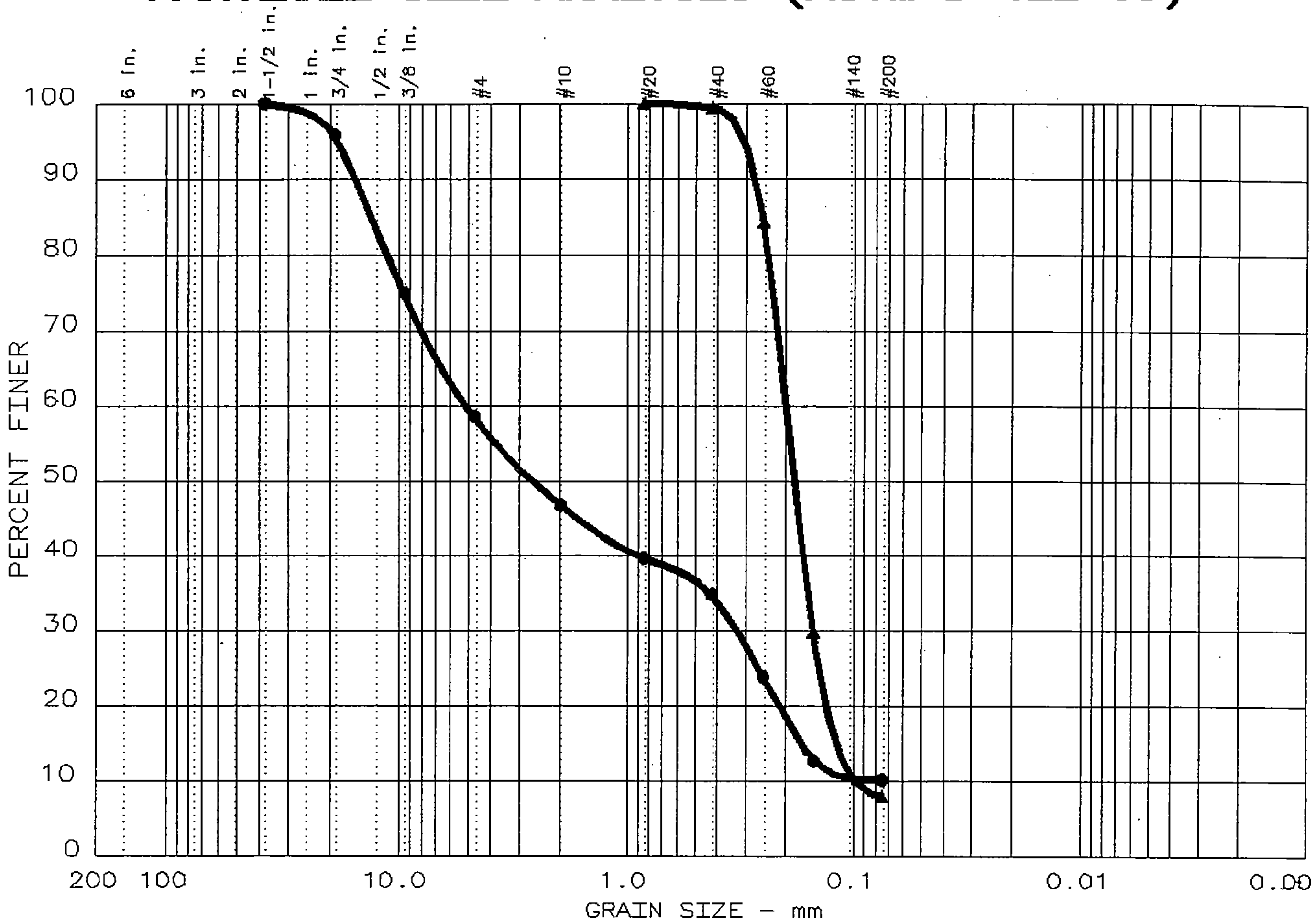
▲ B-11 61-61.5'
Dk.bluish gray silty f-SAND.

Remarks:

Soil Mechanics Lab

Project No.: SF05019
Project: MUNI Power Plant
Date: 8-30-05
Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	41.4	48.4	10.2		SP-SM		
▲	0.0	0.0	92.0	8.0		SP-SM		

SIEVE inches size	PERCENT FINER ●	PERCENT FINER ▲	
1.5	100.0		
0.75	95.8		
0.375	75.0		
	GRAIN SIZE		
D_{60}	5.07	0.20	
D_{30}	0.32	0.15	
D_{10}		0.09	
	COEFFICIENTS		
C_c		1.16	
C_u		2.0	

SIEVE number size	PERCENT FINER ●	PERCENT FINER ▲	
4	58.6		
10	46.8		
20	39.6	100.0	
40	34.8	99.4	
60	23.8	84.3	
100	12.7	29.7	
200	10.2	8.0	

Sample information:

● B-12 15-16'
Dk. gray gravelly SAND w/silt fines.

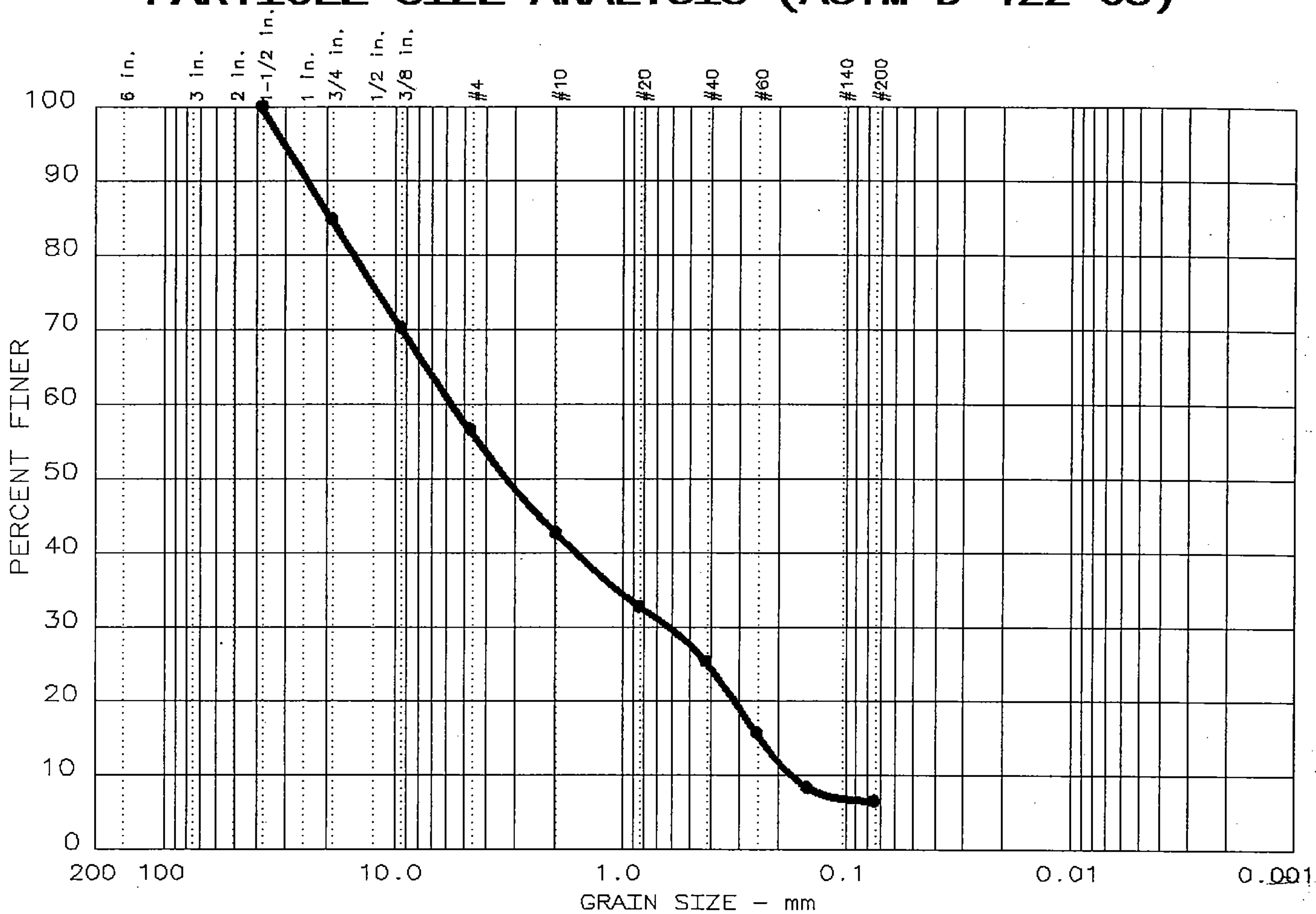
▲ B-12 75-76'
Dk. gray/redish brown f-SAND. Trace silt fines

Remarks:

Soil Mechanics Lab

Project No.: SF05019
Project: MUNI Power Plant
Date: 8-30-05 Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	43.4	50.1	6.5		SO-SM		

SIEVE inches size	PERCENT FINER ●		
1.5	100.0		
0.75	84.8		
0.375	70.3		
	GRAIN SIZE		
D_{60}	5.68		
D_{30}	0.62		
D_{10}	0.17		
	COEFFICIENTS		
C_c	0.39		
C_u	32.7		

SIEVE number size	PERCENT FINER ●		
4	56.6		
10	42.7		
20	32.8		
40	25.5		
60	15.8		
100	8.4		
200	6.5		

Sample information:

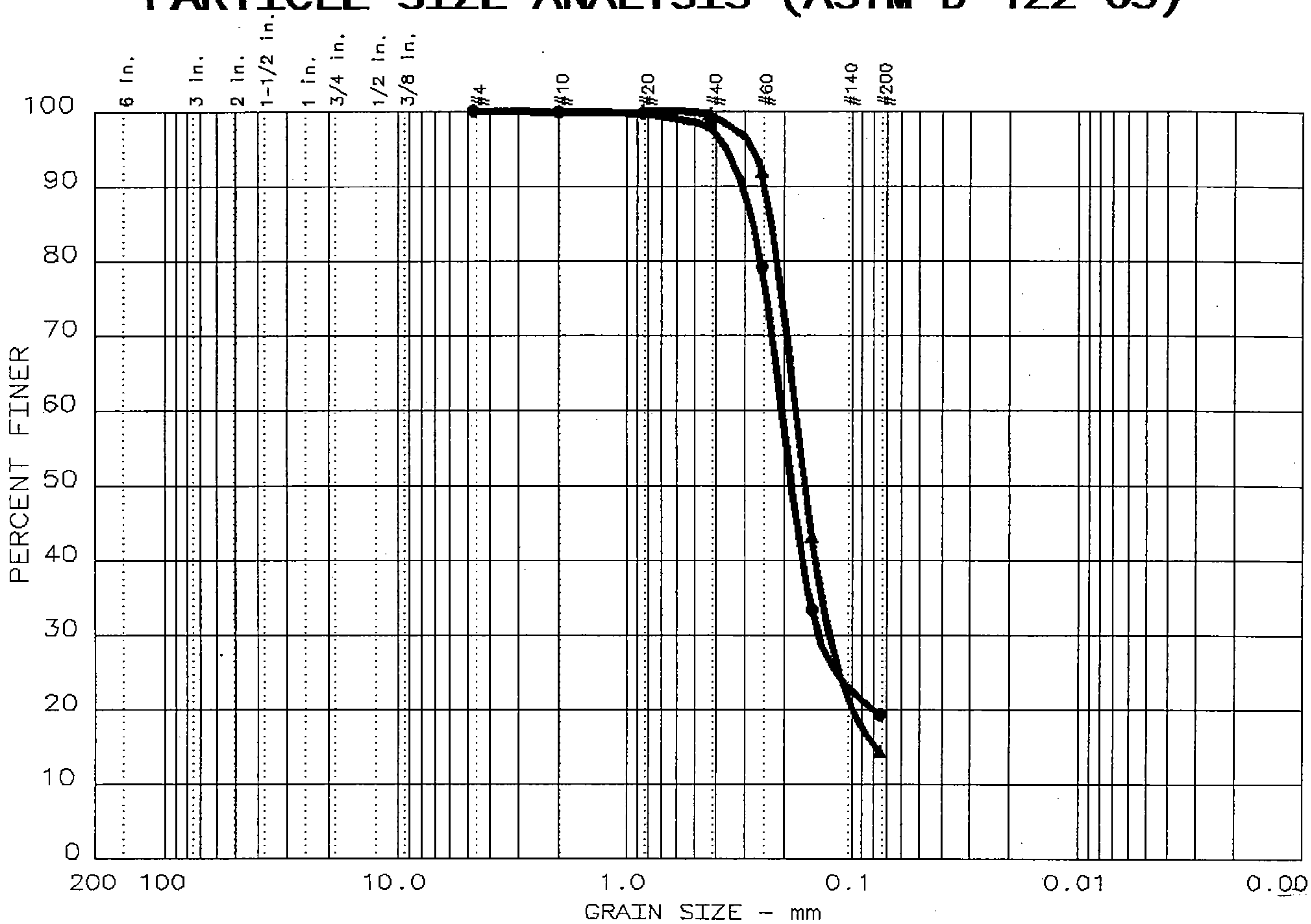
● B-13 16-16.5'
V.dk. gray gravelly
SAND. Trace silt fines.

Remarks:

Soil Mechanics Lab

Project No.: SF05019
Project: MUNI Power Plant
Date: 8-30-05
Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	0.0	80.7	19.3		SM		
▲	0.0	0.0	85.6	14.4		SM		

SIEVE inches size	PERCENT FINER ●	▲	
	GRAIN SIZE		
D_{60}	0.20	0.18	
D_{30}	0.14	0.12	
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ●	▲	
4	100.0		
10	99.8		
20	99.7	100.0	
40	98.0	99.6	
60	79.2	91.8	
100	33.4	43.2	
200	19.3	14.4	

Sample information:

● B-14 65-66.5'
Dk. gray silty SAND.

▲ B-14 80-81.5'
Dk. brn.silty f-SAND.

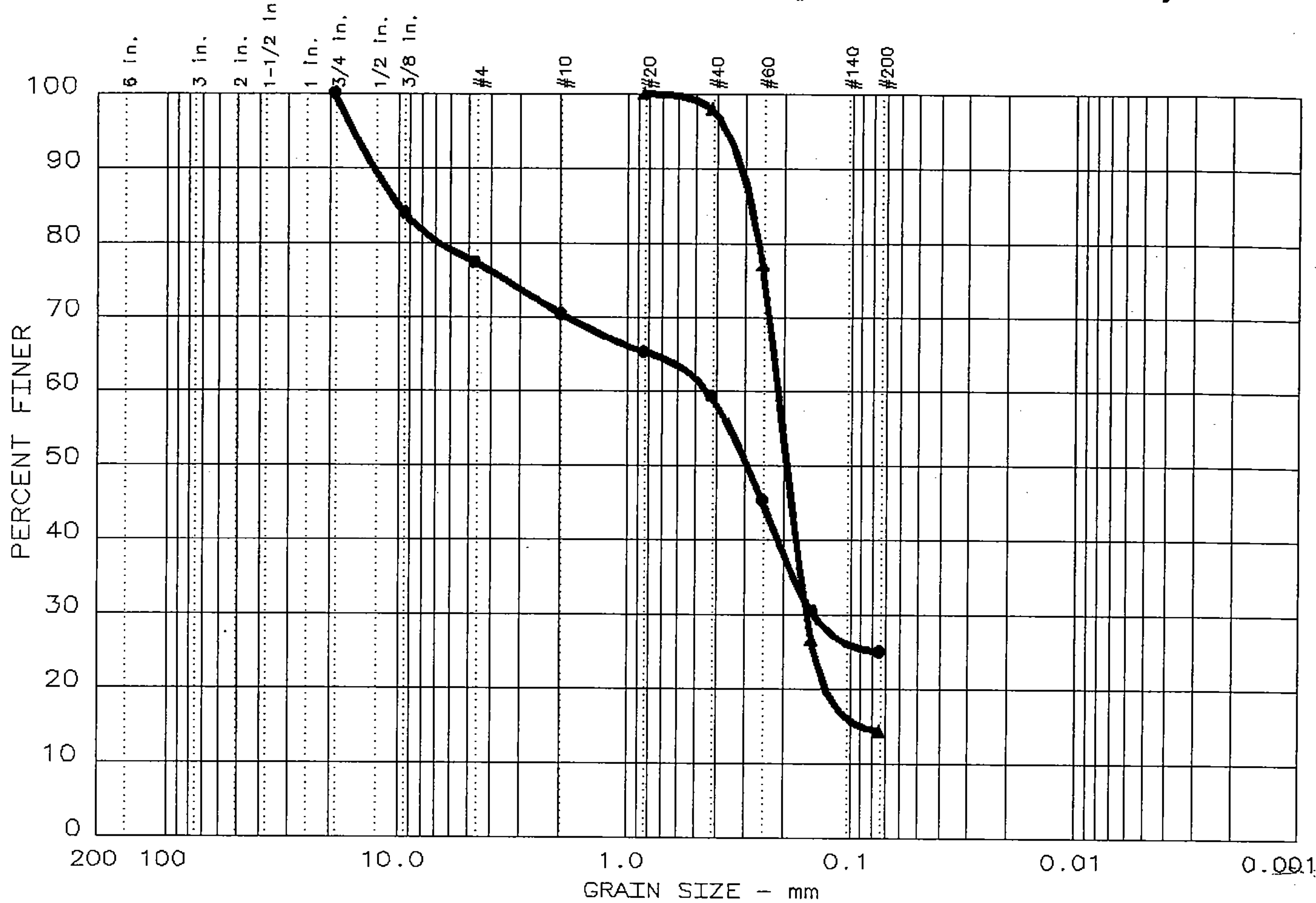
Remarks:

Soil Mechanics Lab

Project No.: SF05019
Project: MUNI Power Plant

Date: 8-30-05 Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	22.6	52.3	25.1		SC		
▲	0.0	0.0	85.6	14.4		SM		

SIEVE inches size	PERCENT FINER ●	▲	
0.75	100.0		
0.375	84.0		
	GRAIN SIZE		
D_{60}	0.44	0.21	
D_{30}	0.15	0.16	
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ●	▲	
4	77.4		
10	70.4		
20	65.3	100.0	
40	59.3	97.9	
60	45.4	77.1	
100	30.5	26.7	
200	25.1	14.4	

Sample information:

● B-15 11-11.5'
Brownish black clayey SAND w/gravel.

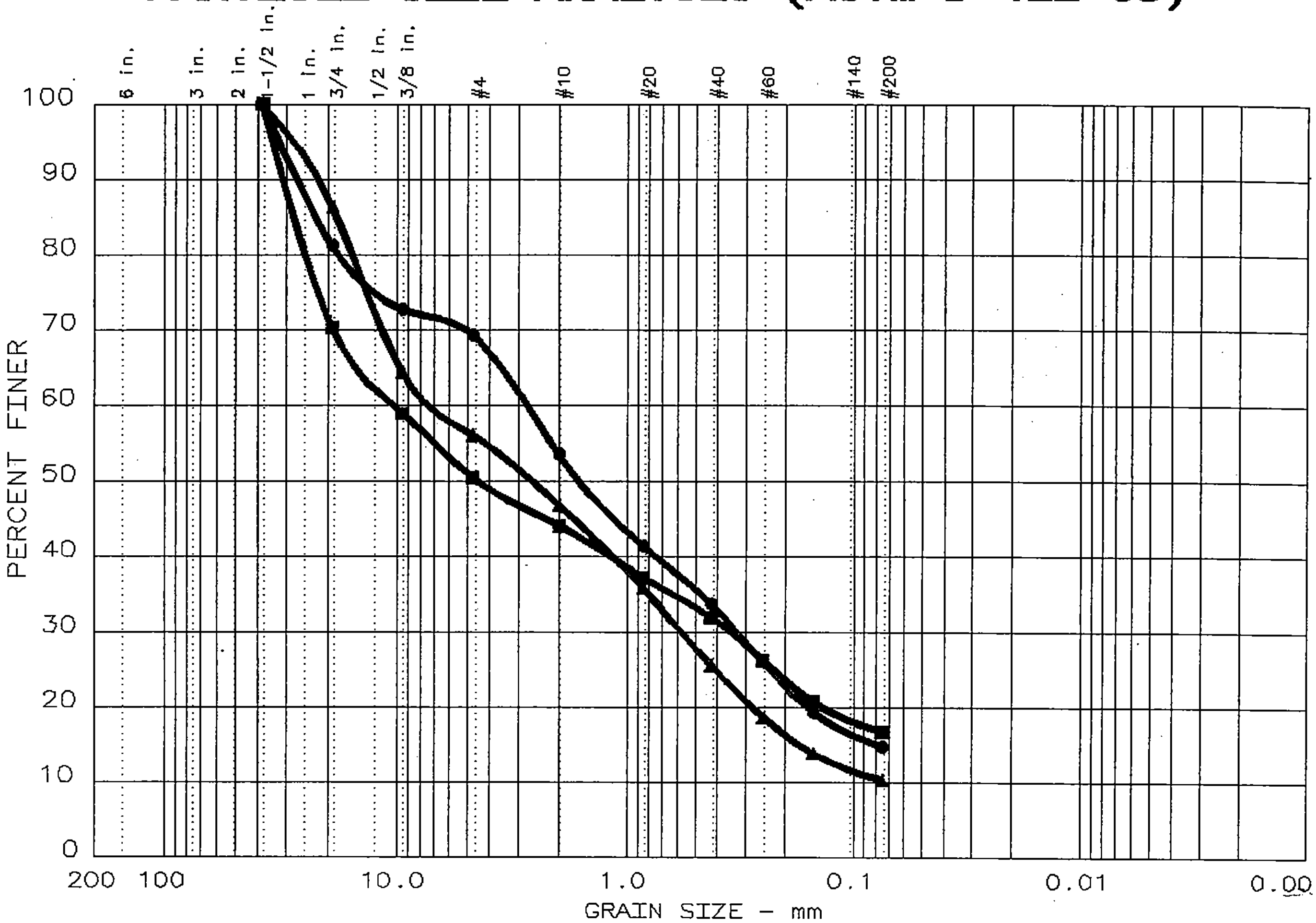
▲ B-15 110-111.5'
Dark gray silty f-SAND.

Remarks:

Soil Mechanics Lab

Project No.: SF05019
Project: MUNI Power Plant
Date: 8-30-05
Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT / % CLAY	USCS	LL	PI
●	0.0	30.7	54.5	14.8	SM		
▲	0.0	44.0	45.7	10.3	SP-SM		
■	0.0	49.6	33.7	16.7	GM		

SIEVE inches size	PERCENT FINER ●	▲	■
1.5	100.0	100.0	100.0
0.75	81.2	86.3	70.2
0.375	72.7	64.3	58.9
	GRAIN SIZE		
D_{60}	2.75	7.41	10.47
D_{30}	0.32	0.57	0.35
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ●	▲	■
4	69.3	56.0	50.4
10	53.5	46.8	44.1
20	41.4	35.8	37.2
40	33.7	25.6	31.9
60	26.1	18.7	26.3
100	19.4	13.8	20.8
200	14.7	10.3	16.7

Sample information:

● B-2 2-5'
Dark brown gravelly SAND w/silt fines.

▲ B-4 1-4'
Gray gravelly SAND. Trace silt fines.

■ B-7 2-5'
Dark to redish brown sandy GRAVEL w/bricks.

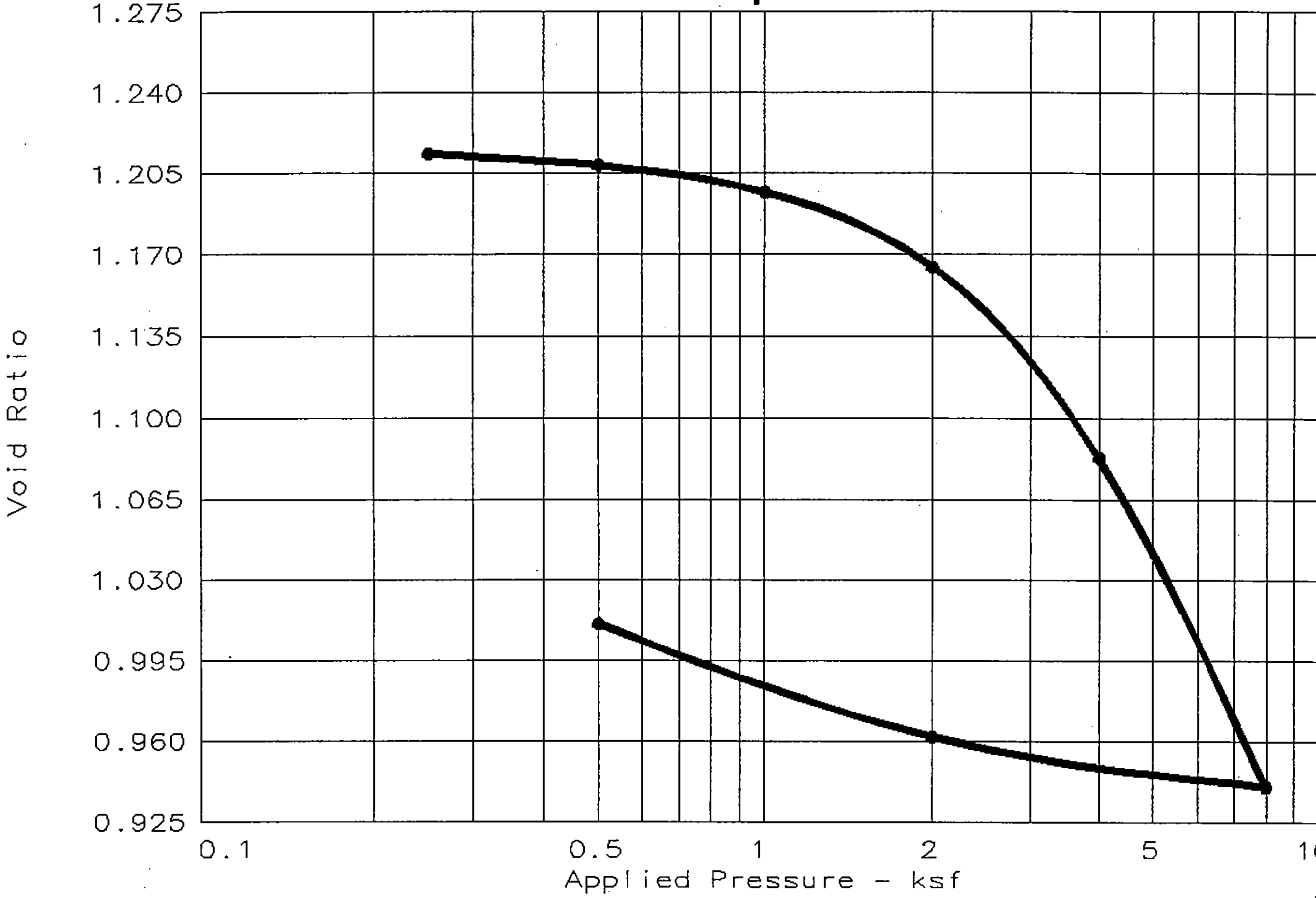
Remarks:

Soil Mechanics Lab

Project No.: SF05019
Project: MUNI Power Plant

Date: 9-5-05 Data Sheet No. ____

Consolidation Test Report-ASTM D 2435



Coeffs. of Consolidation (sq. ft./day) & Secondary Consolidation											
No.	Load	Cv	Cα	No.	Load	Cv	Cα	No.	Load	Cv	Cα
5	4.00	0.04	0.016								

Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Precons. (ksf)	C_c	e_o
90.6 %	41.2 %	75.7			2.700	3.10	0.49	1.2269

TEST RESULTS	MATERIAL DESCRIPTION
Compression Index = 0.49	Med.stiff,,dark gray FAT CLAY w/shell frags.
	Class: CH

Project No.: SF05019
Project: Muni Power Plant
Location: B-1 34-37'
Test @ 36.5'
Date: 9-1-05

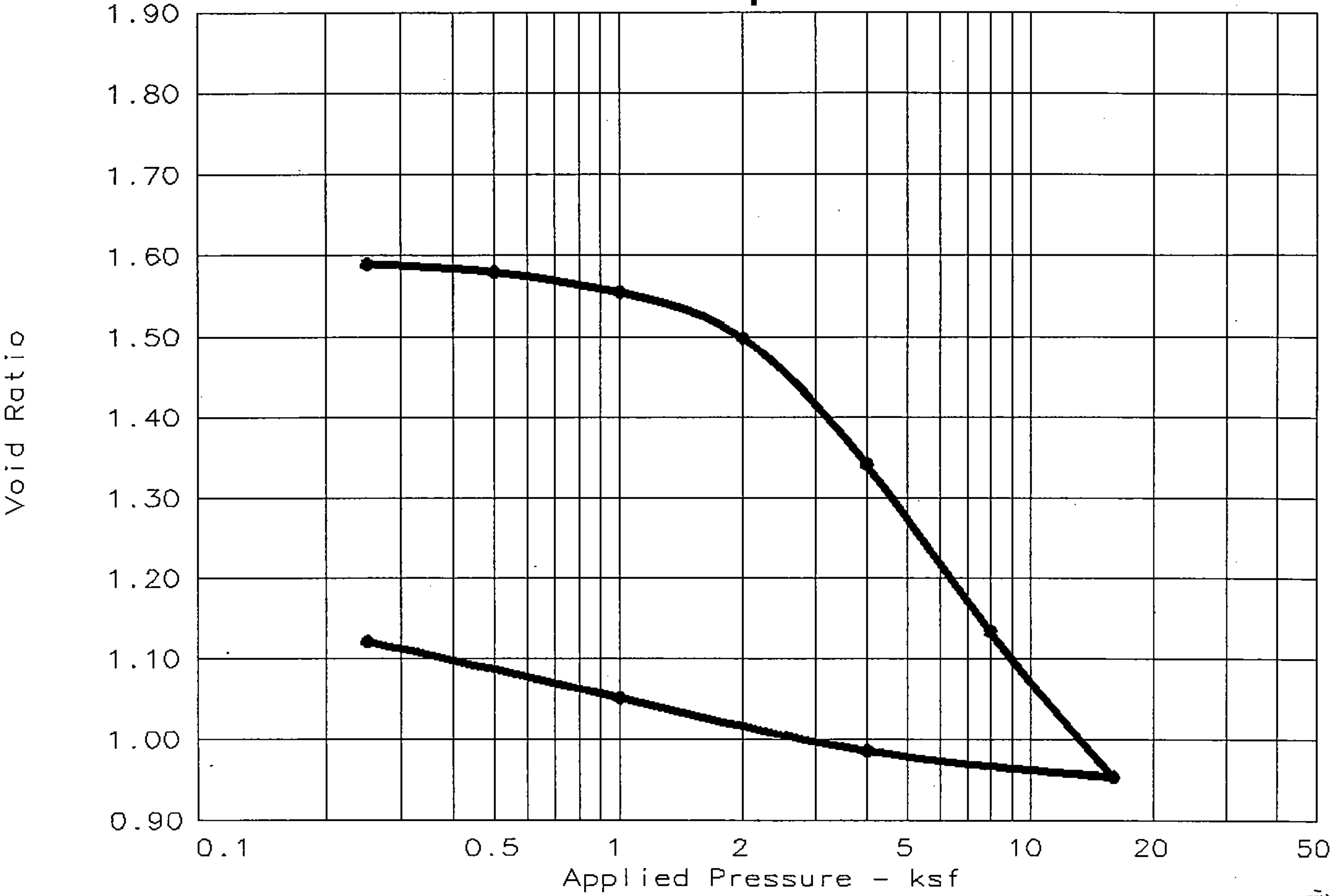
Remarks:

Consolidation Test Report-ASTM D 2435

Soil Mechanics Lab

Fig. No. ______

Consolidation Test Report-ASTM D 2435



Coeffs. of Consolidation (sq. ft./day) & Secondary Consolidation											
No.	Load	Cv	Cα	No.	Load	Cv	Cα	No.	Load	Cv	Cα
5	4.00	0.01	0.012								
6	8.00	0.01	0.013								

Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Precons. (ksf)	C_c	e_o
94.8 %	55.8 %	65.1	73	44	2.700	2.30	0.68	1.5892

TEST RESULTS

Compression Index = 0.68

MATERIAL DESCRIPTION

Soft, dark gray FAT CLAY.

Class: CH

Remarks:

Cc is btwn. 2 & 4 ksf.

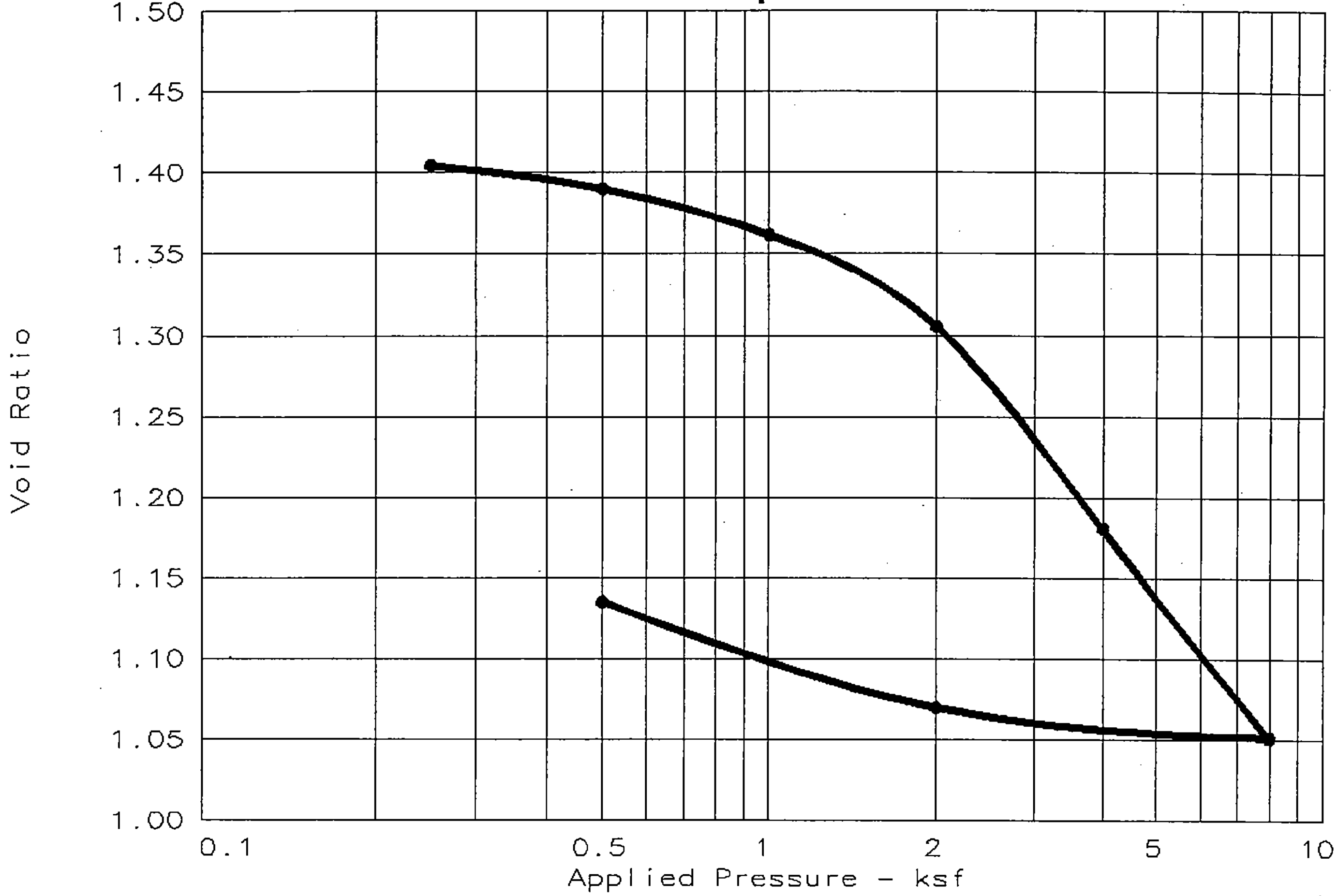
Project No.: SF05019
Project: Muni Power Plant
Location: B-2 35-38'
Test @ 37.5'
Date: 9-1-05

Consolidation Test Report-ASTM D 2435

Soil Mechanics Lab

Fig. No. ______

Consolidation Test Report-ASTM D 2435



Coeffs. of Consolidation (sq. ft./day) & Secondary Consolidation											
No.	Load	Cv	Cα	No.	Load	Cv	Cα	No.	Load	Cv	Cα
4	2.00	0.08	0.010								
5	4.00	0.02	0.016								

Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Precons. (ksf)	C_c	e_o
94.3 %	49.4 %	69.8			2.700	1.90	0.43	1.4138

TEST RESULTS

Compression Index = 0.43

MATERIAL DESCRIPTION

Med.stiff,,dark gray FAT CLAY w/shell frags.

Class: CH

Remarks:

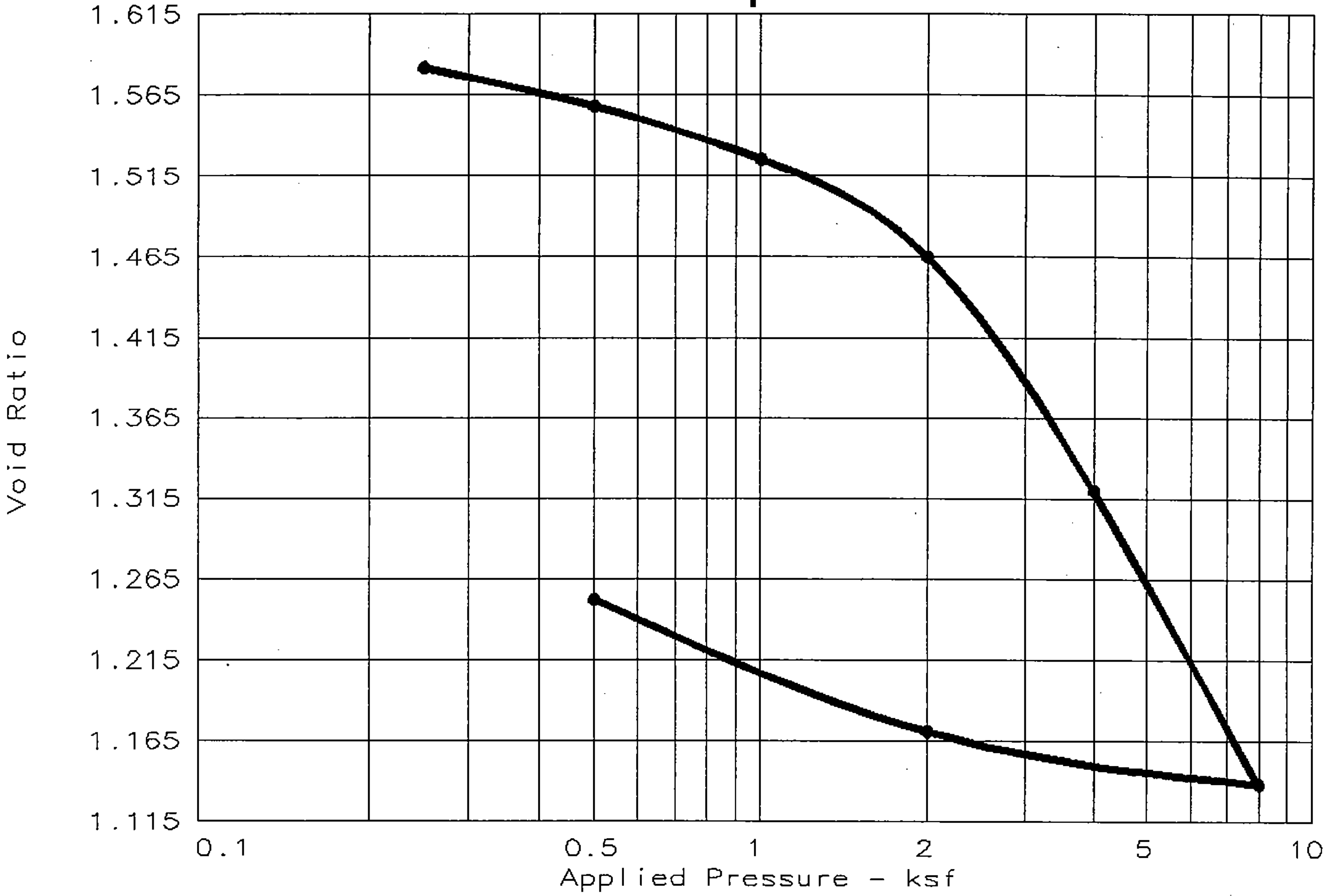
Project No.: SF05019
Project: Muni Power Plant
Location: B-4 34-37'
Test @ 36.5'
Date: 9-1-05

Consolidation Test Report-ASTM D 2435

Soil Mechanics Lab

Fig. No. ______

Consolidation Test Report-ASTM D 2435



Coeffs. of Consolidation (sq. ft./day) & Secondary Consolidation											
No.	Load	Cv	Cα	No.	Load	Cv	Cα	No.	Load	Cv	Cα
4	2.00	0.02	0.008								
5	4.00	0.01	0.014								

Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Precons. (ksf)	C_c	e_o
94.6 %	55.9 %	65.0			2.700	2.30	0.60	1.5948

TEST RESULTS	MATERIAL DESCRIPTION
Compression Index = 0.60	Soft,dark gray FAT CLAY Trace(-)shell frags.
	Class: CH

Project No.: SF05019
Project: Muni Power Plant
Location: B-5 40-43'
Test @ 42.5'
Date: 9-1-05

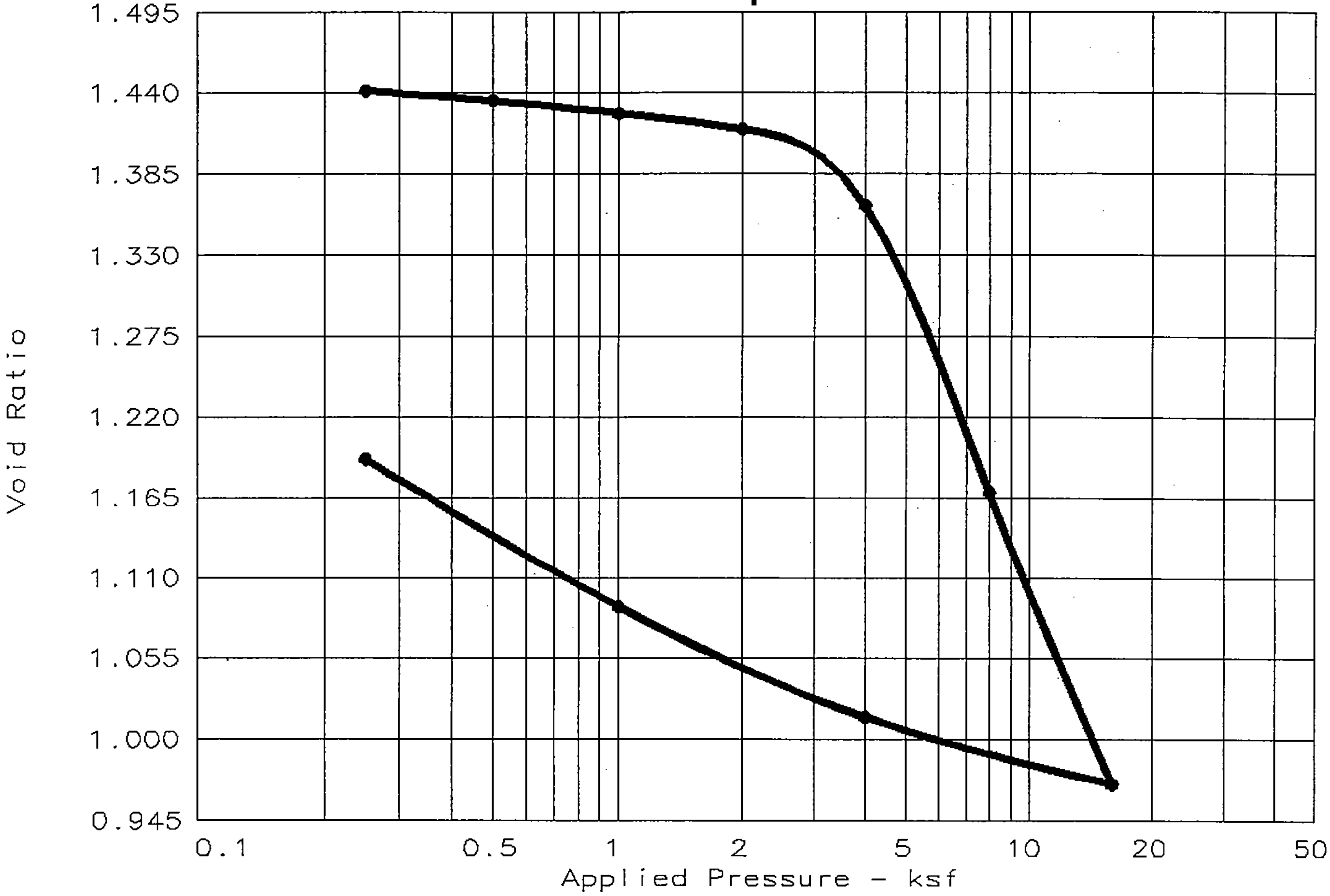
Remarks:

Consolidation Test Report-ASTM D 2435

Soil Mechanics Lab

Fig. No. ______

Consolidation Test Report-ASTM D 2435



Coeffs. of Consolidation (sq. ft./day) & Secondary Consolidation											
No.	Load	Cv	Cα	No.	Load	Cv	Cα	No.	Load	Cv	Cα
5	4.00	0.32	0.005								
6	8.00	0.02	0.018								

Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Precons. (ksf)	C_c	e_o
97.1 %	51.8 %	69.1	75	45	2.700	4.00	0.70	1.4407

TEST RESULTS

Compression Index = 0.70

MATERIAL DESCRIPTION

Med.stiff,dk. gray FAT CLAY.

Class: CH

Remarks:

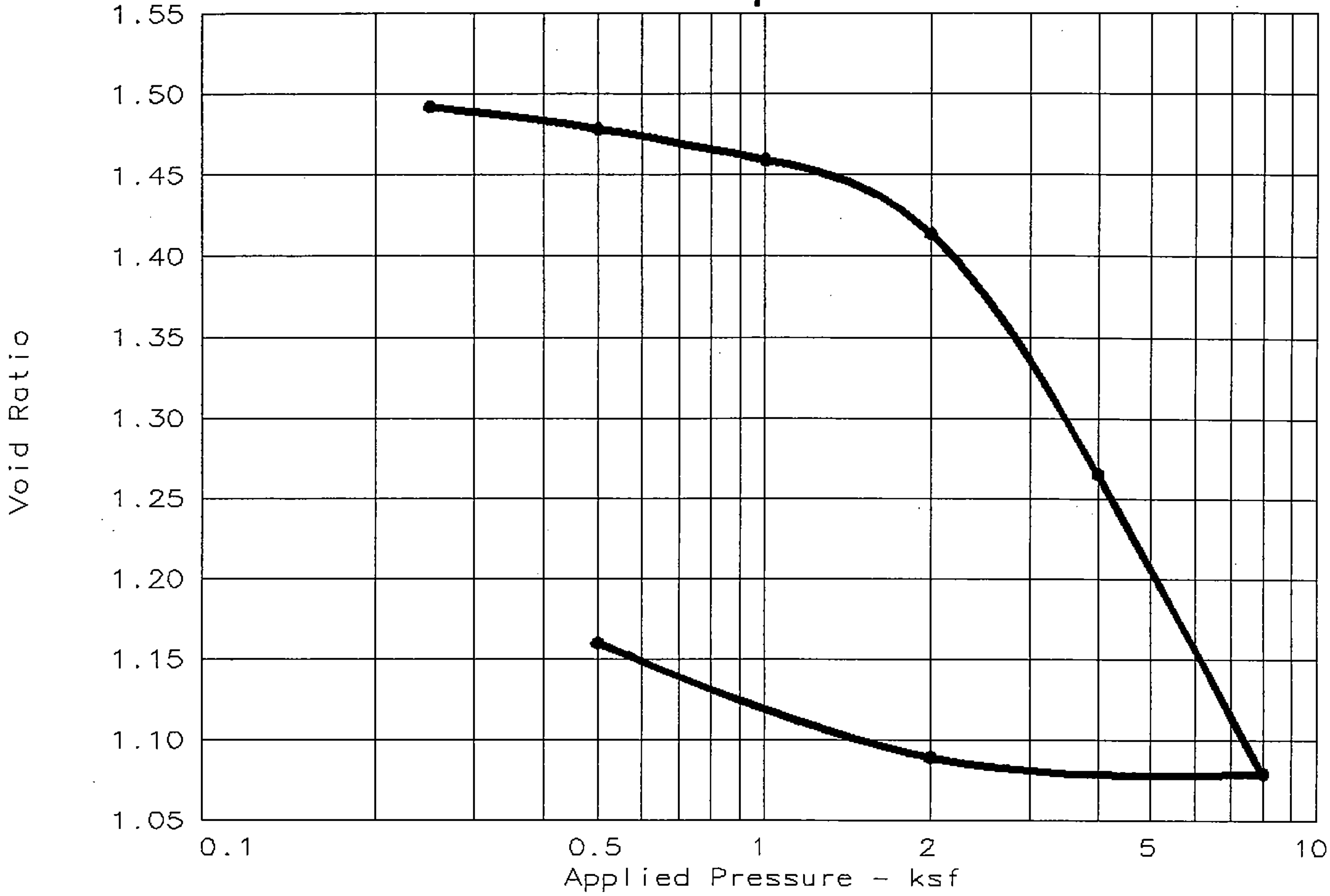
Project No.: SF05019
Project: Muni Power Plant
Location: B-6 49-52'
Test @ 51.5'
Date: 9-1-05

Consolidation Test Report-ASTM D 2435

Soil Mechanics Lab

Fig. No. ______

Consolidation Test Report-ASTM D 2435



Coeffs. of Consolidation (sq. ft./day) & Secondary Consolidation											
No.	Load	Cv	Cα	No.	Load	Cv	Cα	No.	Load	Cv	Cα
4	2.00	0.12	0.005								
5	4.00	0.01	0.017								

Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Precons. (ksf)	C_c	e_o
94.2 %	52.2 %	67.5	68	40	2.700	2.20	0.62	1.4960

TEST RESULTS	MATERIAL DESCRIPTION
Compression Index = 0.62	Med.stiff,dk. gray FAT CLAY w/c-shell frags.
	Class: CH

Project No.: SF05019
Project: Muni Power Plant
Location: B-7 30-33'
Test @ 32.5'
Date: 9-1-05

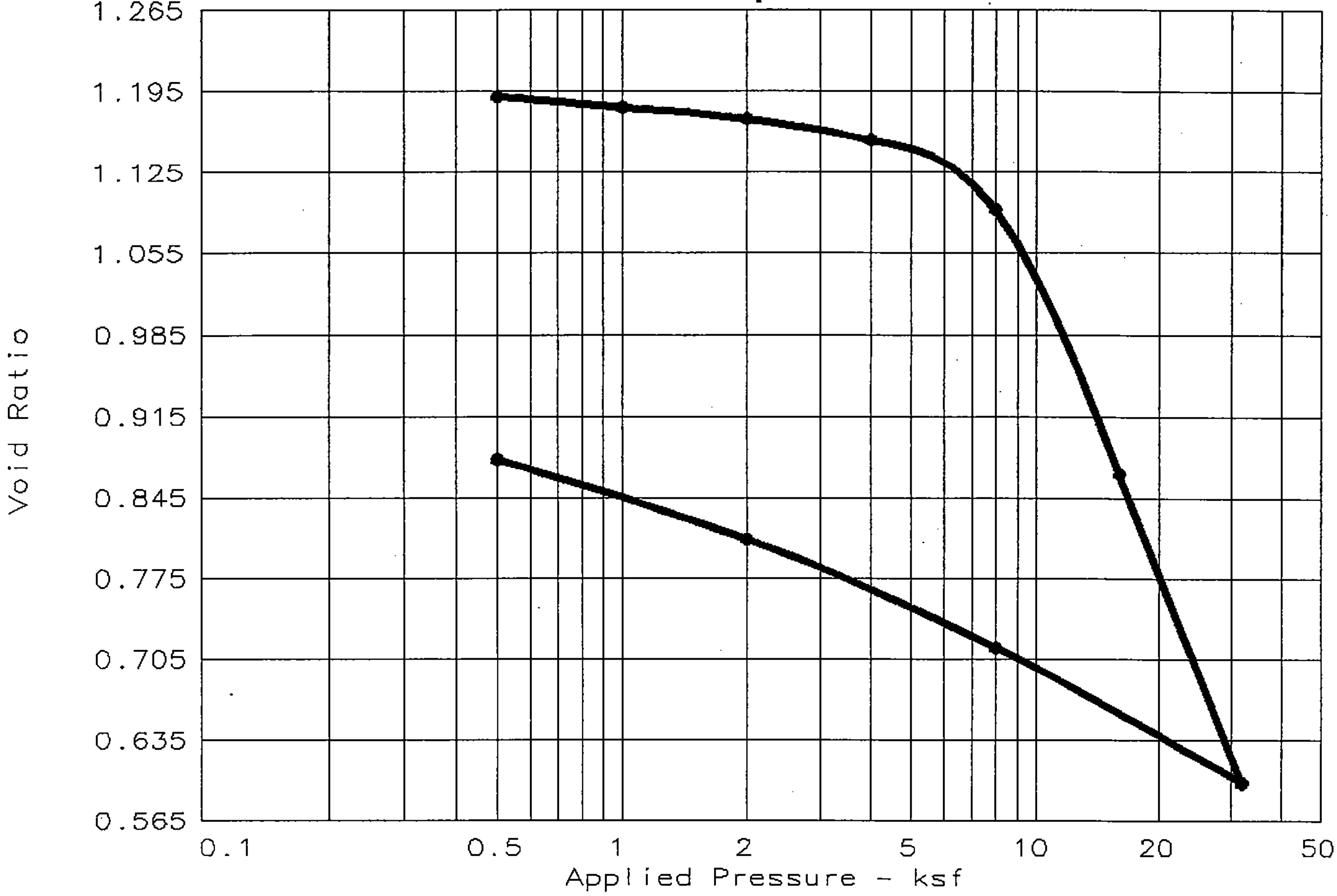
Remarks:

Consolidation Test Report-ASTM D 2435

Soil Mechanics Lab

Fig. No. ______

Consolidation Test Report-ASTM D 2435



Coeffs. of Consolidation (sq. ft./day) & Secondary Consolidation											
No.	Load	Cv	Cα	No.	Load	Cv	Cα	No.	Load	Cv	Cα
5	8.00	0.09	0.007								
6	16.00	0.01	0.018								

Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Precons. (ksf)	C_c	e_o
113.4 %	50.0 %	77.0	75	50	2.700	8.47	0.91	1.1904

TEST RESULTS	MATERIAL DESCRIPTION
Compression Index = 0.91	Stiff, dark gray FAT CLAY. Class: CH

Project No.: SF05019
Project: Muni Power Plant
Location: B-8 90-93'
Test @ 92.5'
Date: 7-11-05

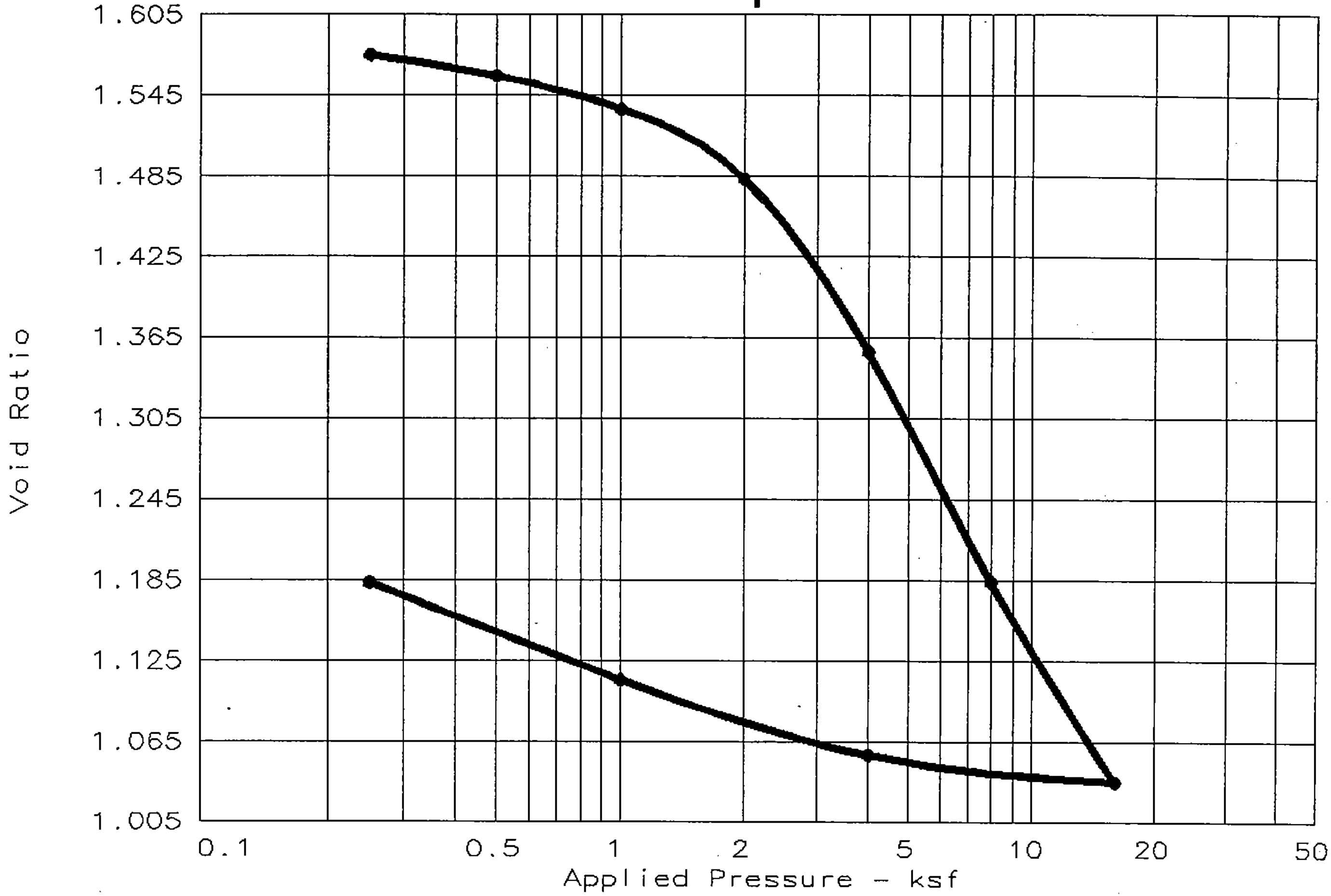
Remarks:

Consolidation Test Report-ASTM D 2435

Soil Mechanics Lab

Fig. No. ______

Consolidation Test Report-ASTM D 2435



Coeffs. of Consolidation (sq. ft./day) & Secondary Consolidation											
No.	Load	Cv	Cα	No.	Load	Cv	Cα	No.	Load	Cv	Cα
5	4.00	0.02	0.015								
6	8.00	0.02	0.012								

Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Precons. (ksf)	C_c	e_o
101.8 %	59.4 %	65.5			2.700	2.40	0.58	1.5751

TEST RESULTS

Compression Index = 0.58

Project No.: SF05019
Project: Muni Power Plant
Location: B-9 28-31'
Test @ 30.5'
Date: 9-1-05

Consolidation Test Report-ASTM D 2435

Soil Mechanics Lab

MATERIAL DESCRIPTION

Soft, dk. gray FAT CLAY. Trace(-)shell frags.

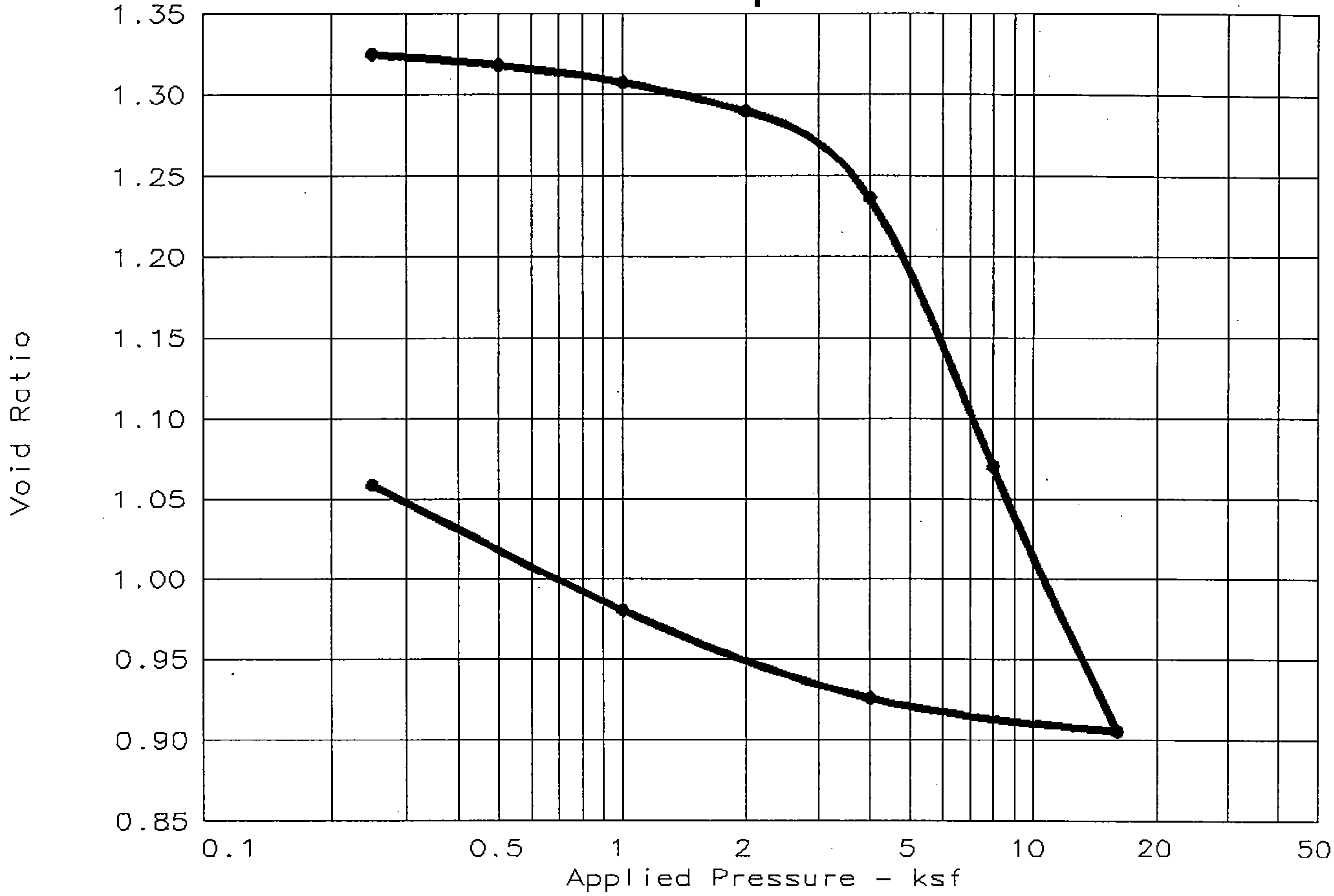
Class: CH

Remarks:

Cc is btwn. 4 & 8 ksf.

Fig. No. ______

Consolidation Test Report-ASTM D 2435



Coeffs. of Consolidation (sq. ft./day) & Secondary Consolidation											
No.	Load	Cv	Cα	No.	Load	Cv	Cα	No.	Load	Cv	Cα
6	8.00	0.02	0.019								
7	16.00	0.03	0.013								

Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Precons. (ksf)	C_c	e_o
98.7 %	48.4 %	72.5	67	41	2.700	3.80	0.60	1.3244

TEST RESULTS	MATERIAL DESCRIPTION
Compression Index = 0.60	Med.stiff,dk.gray FAT CLAY. Class: CH

Project No.: SF05019
Project: Muni Power Plant
Location: B-11 45-48'
Test @ 46.5'
Date: 9-1-05

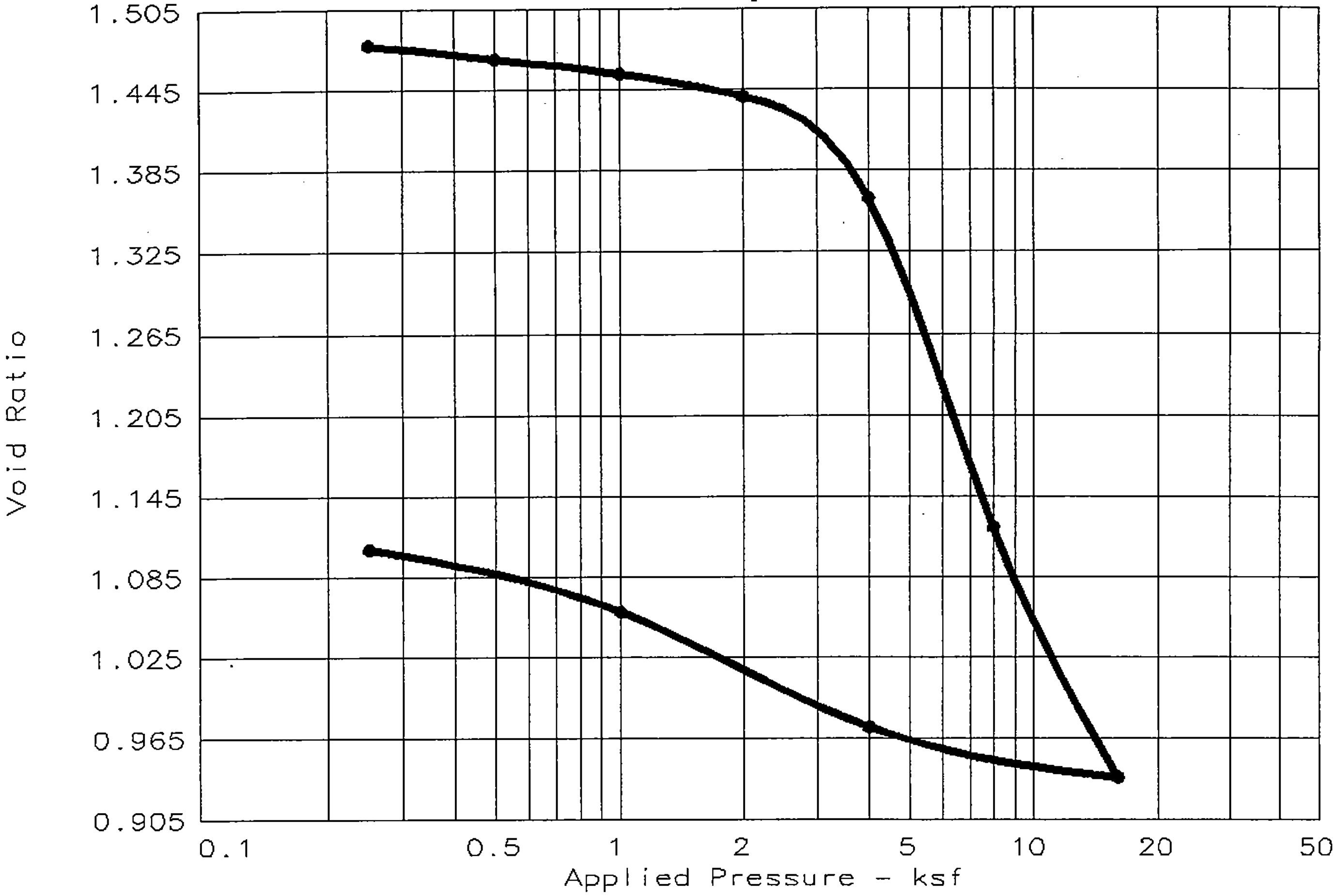
Remarks:

Consolidation Test Report-ASTM D 2435

Soil Mechanics Lab

Fig. No. ______

Consolidation Test Report-ASTM D 2435



Coeffs. of Consolidation (sq. ft./day) & Secondary Consolidation											
No.	Load	Cv	Cα	No.	Load	Cv	Cα	No.	Load	Cv	Cα
6	8.00	0.01	0.011								
7	16.00	0.01	0.018								

Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Precons. (ksf)	C_c	e_o
97.5 %	53.4 %	68.0	70	44	2.700	3.60	0.84	1.4791

TEST RESULTS	MATERIAL DESCRIPTION
Compression Index = 0.84	Med.stiff,dk.gray FAT CLAY. Class: CH

Project No.: SF05019
Project: Muni Power Plant
Location: B-12 40-43'
Test @ 41.5'
Date: 9-1-05

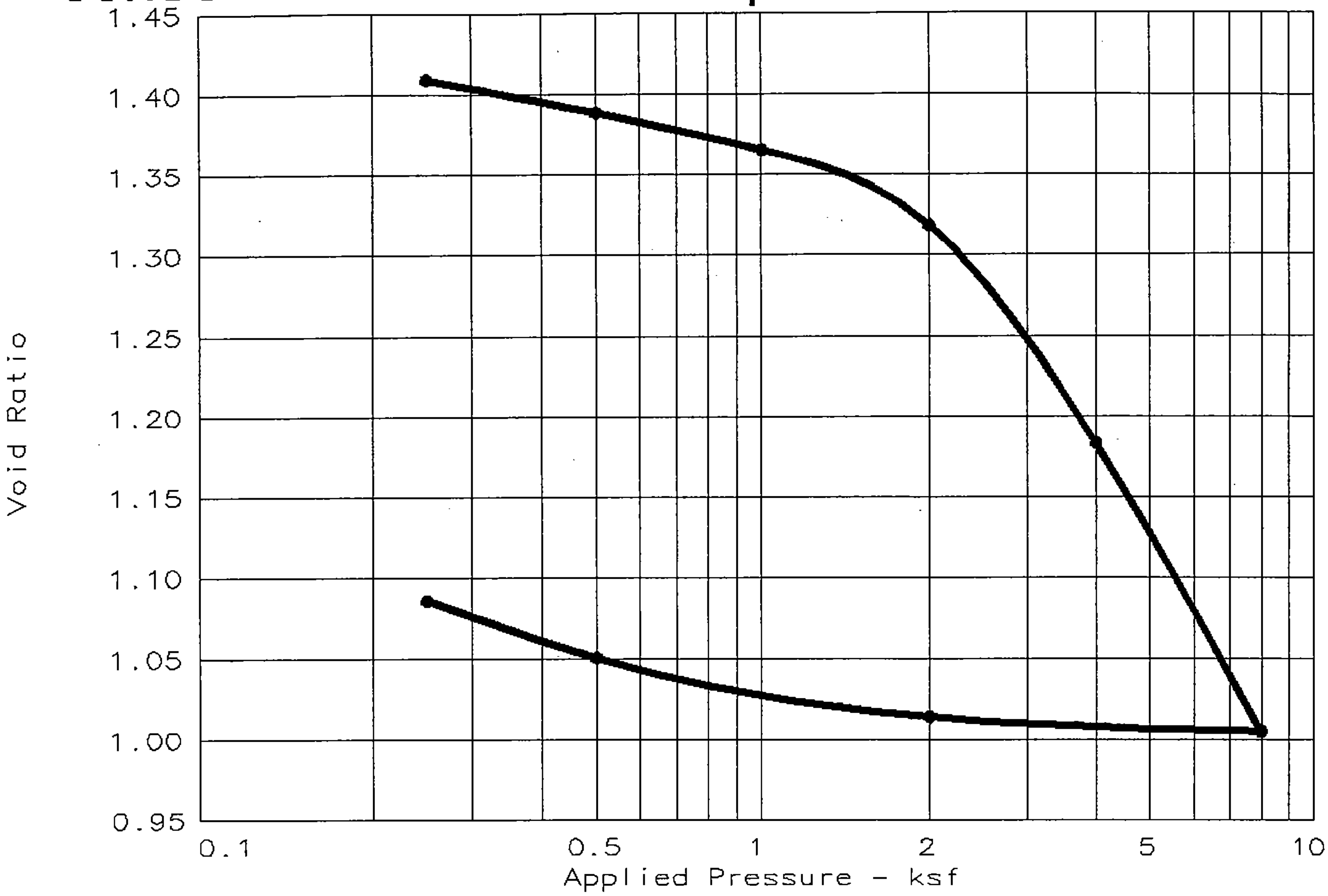
Remarks:
Cc is btwn. 4 & 8 ksf.

Consolidation Test Report-ASTM D 2435

Soil Mechanics Lab

Fig. No. ______

Consolidation Test Report-ASTM D 2435



Coeffs. of Consolidation (sq. ft./day) & Secondary Consolidation											
No.	Load	Cv	Cα	No.	Load	Cv	Cα	No.	Load	Cv	Cα
5	4.00	0.02	0.027								

Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Precons. (ksf)	C_c	e_o
97.5 %	51.9 %	69.1	65	38	2.700	2.40	0.61	1.4377

TEST RESULTS	MATERIAL DESCRIPTION
Compression Index = 0.61	Med.stiff,dk.gray FAT CLAY.Trace shell frags. Class: CH

Project No.: SF05019
Project: Muni Power Plant
Location: B-14 35-38'
Test @ 37.5'
Date: 9-1-05

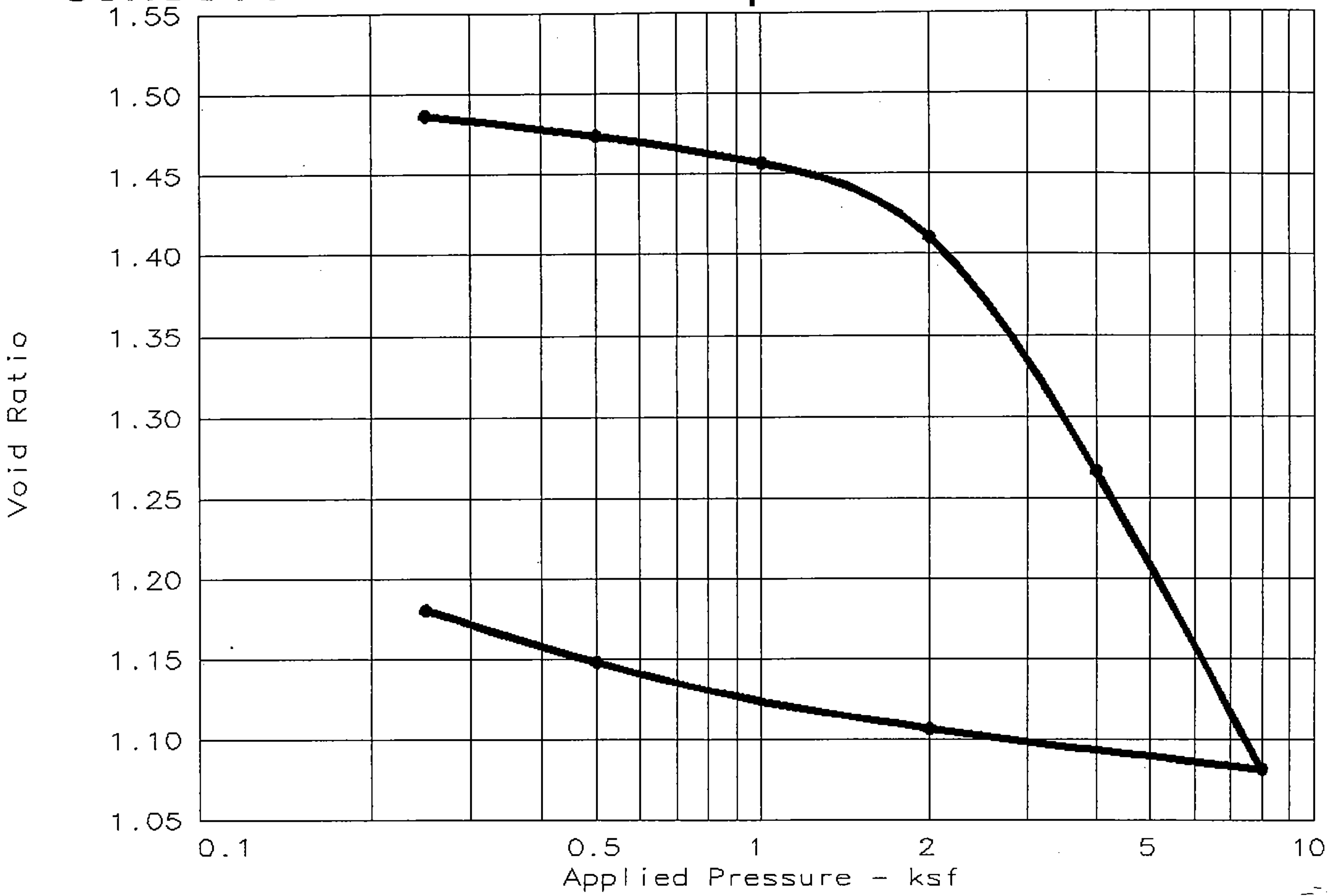
Remarks:

Consolidation Test Report-ASTM D 2435

Soil Mechanics Lab

Fig. No. ______

Consolidation Test Report-ASTM D 2435



Coeffs. of Consolidation (sq. ft./day) & Secondary Consolidation											
No.	Load	Cv	Cα	No.	Load	Cv	Cα	No.	Load	Cv	Cα
4	2.00	0.10	0.009								
5	4.00	0.01	0.028								

Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Precons. (ksf)	C_c	e_o
93.2 %	51.4 %	67.7	72	45	2.700	2.40	0.61	1.4884

TEST RESULTS	MATERIAL DESCRIPTION
Compression Index = 0.61	Soft,dk.gray FAT CLAY w/shell fragments. Class: CH

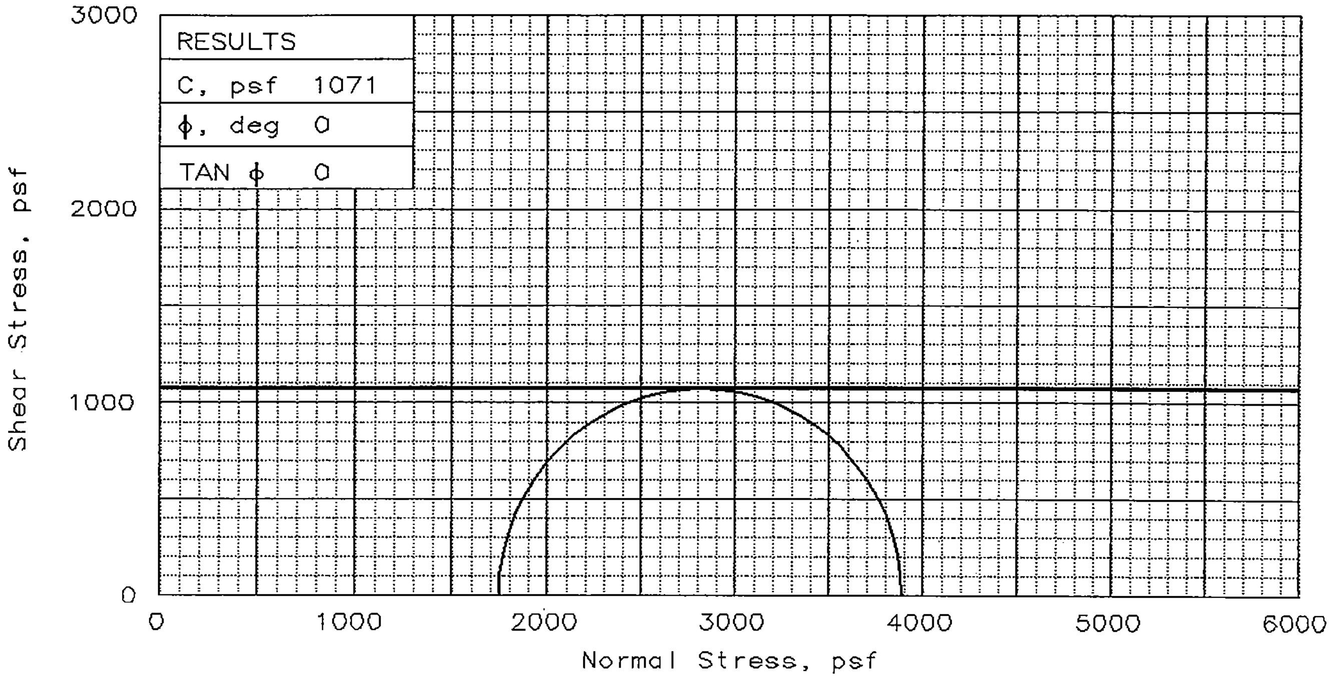
Project No.: SF05019
Project: Muni Power Plant
Location: B-15 30-33'
Test @ 32.0'
Date: 9-1-05

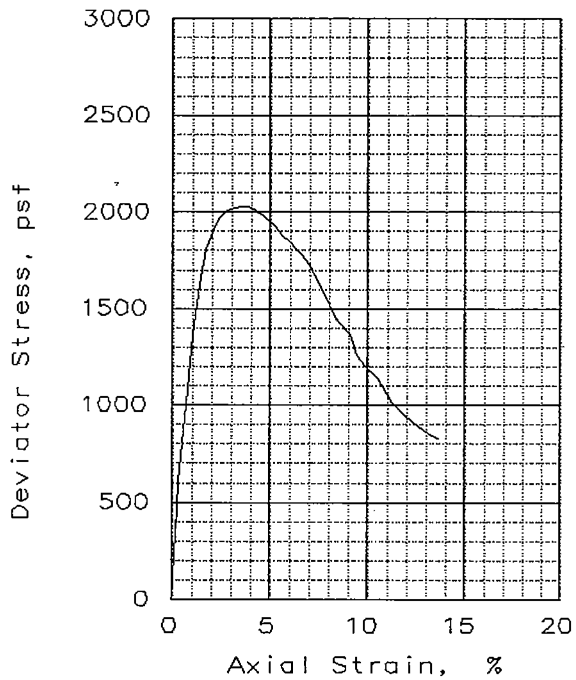
Remarks:

Consolidation Test Report-ASTM D 2435

Soil Mechanics Lab

Fig. No. ______





SAMPLE NO.:		1
INITIAL	WATER CONTENT, %	45.2
	DRY DENSITY, pcf	74.0
	SATURATION, %	95.5
	VOID RATIO	1.278
	DIAMETER, in	2.88
	HEIGHT, in	5.70
AT TEST	WATER CONTENT, %	45.2
	DRY DENSITY, pcf	74.0
	SATURATION, %	95.5
	VOID RATIO	1.278
	DIAMETER, in	2.88
	HEIGHT, in	5.70
Strain rate, in/min		0.0750
BACK PRESSURE, psf		0
CELL PRESSURE, psf		1750
FAIL. STRESS, psf		2141
STRAIN, %		3.9
ULT. STRESS, psf		
STRAIN, %		
σ_1 FAILURE, psf		3891
σ_3 FAILURE, psf		1750

TYPE OF TEST:
Unconsolidated Undrained

SAMPLE TYPE: Shelby

DESCRIPTION: Stiff,dk.gray FAT CLAY(CH)-Trace shell frags.

SPECIFIC GRAVITY= 2.7

REMARKS:

Fig. No.: ______

CLIENT: Geotechnical Consultants,Inc.

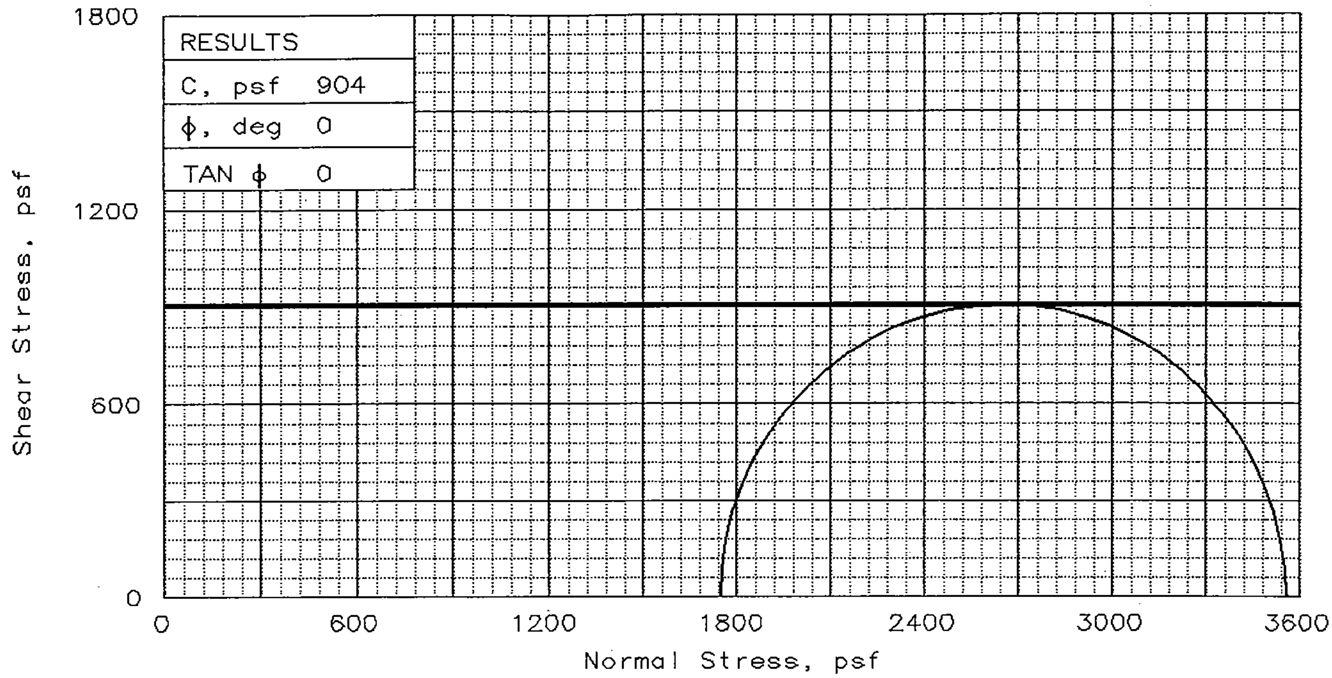
PROJECT: MUNI Power Plant

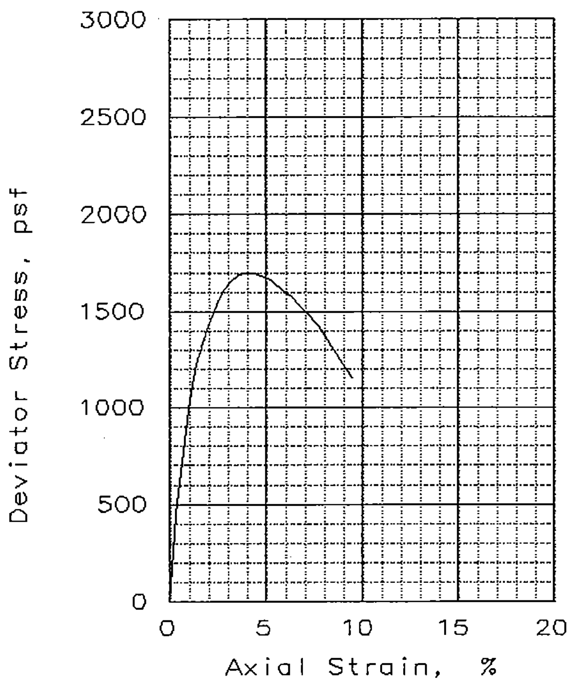
SAMPLE LOCATION: B-1 34-37'
Test @ 35'

PROJ. NO.: SF05019 DATE: 8-23-05

TRIAXIAL SHEAR TEST REPORT

Soil Mechanics Lab





SAMPLE NO.:		1
INITIAL	WATER CONTENT, %	49.3
	DRY DENSITY, pcf	70.7
	SATURATION, %	96.1
	VOID RATIO	1.384
	DIAMETER, in	2.88
	HEIGHT, in	5.70
AT TEST	WATER CONTENT, %	49.3
	DRY DENSITY, pcf	70.7
	SATURATION, %	96.1
	VOID RATIO	1.384
	DIAMETER, in	2.88
	HEIGHT, in	5.70
Strain rate, in/min		0.0750
BACK PRESSURE, psf		0
CELL PRESSURE, psf		1750
FAIL. STRESS, psf		1808
STRAIN, %		4.2
ULT. STRESS, psf		
STRAIN, %		
σ_1 FAILURE, psf		3558
σ_3 FAILURE, psf		1750

TYPE OF TEST:
Unconsolidated Undrained
SAMPLE TYPE: Shelby
DESCRIPTION: Stiff,dk.gray FAT CLAY(CH)-Trace shell frags.

SPECIFIC GRAVITY= 2.7
REMARKS:

Fig. No.: ______

CLIENT: Geotechnical Consultants,Inc.

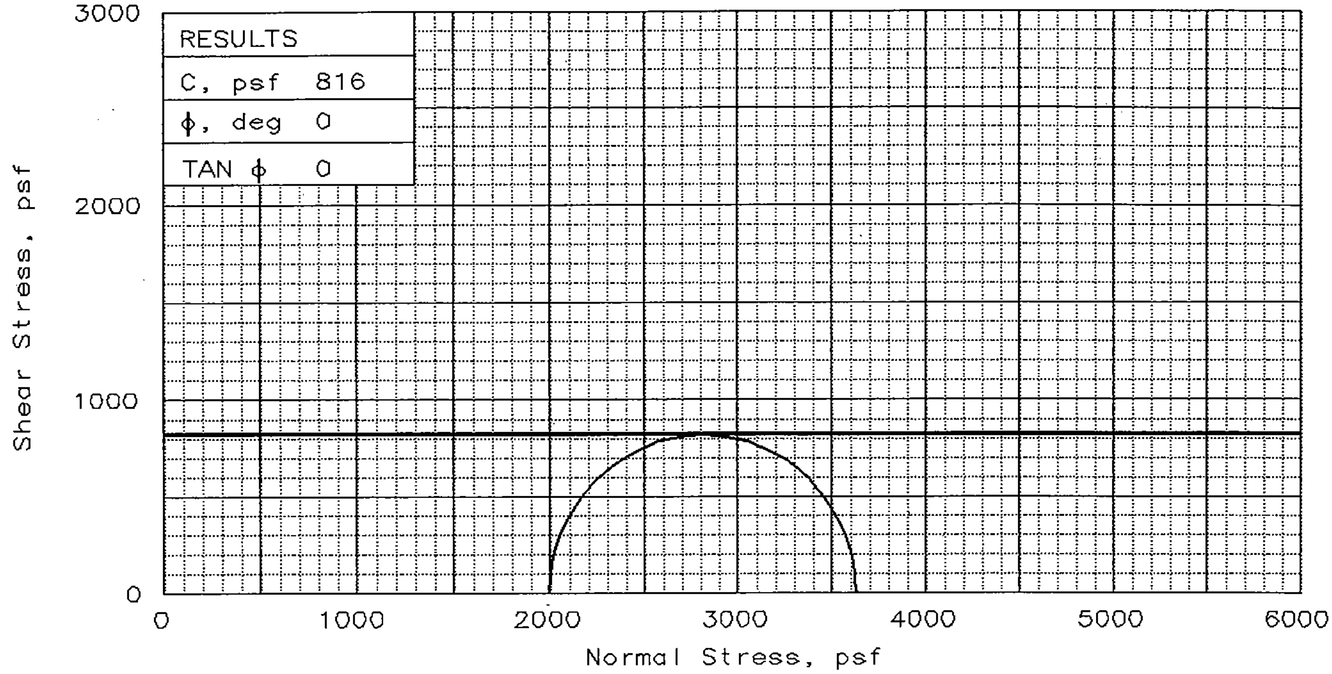
PROJECT: MUNI Power Plant

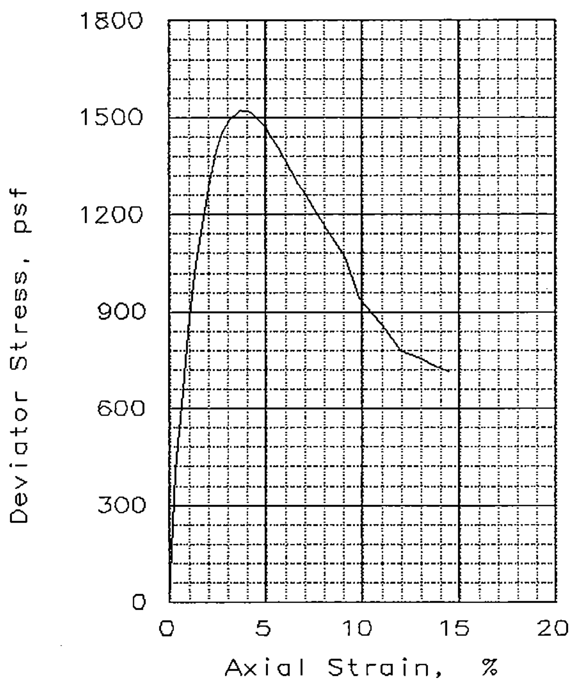
SAMPLE LOCATION: B-4 34-37'
Test @ 35'

PROJ. NO.: SF05019 DATE: 8-23-05

TRIAXIAL SHEAR TEST REPORT

Soil Mechanics Lab





SAMPLE NO.:		1
INITIAL	WATER CONTENT, %	52.0
	DRY DENSITY, pcf	67.7
	SATURATION, %	94.3
	VOID RATIO	1.489
	DIAMETER, in	2.88
	HEIGHT, in	5.68
AT TEST	WATER CONTENT, %	52.0
	DRY DENSITY, pcf	67.7
	SATURATION, %	94.3
	VOID RATIO	1.489
	DIAMETER, in	2.88
	HEIGHT, in	5.68
Strain rate, in/min		0.0750
BACK PRESSURE, psf		0
CELL PRESSURE, psf		2000
FAIL. STRESS, psf		1632
STRAIN, %		3.7
ULT. STRESS, psf		
STRAIN, %		
σ_1 FAILURE, psf		3632
σ_3 FAILURE, psf		2000

TYPE OF TEST:
Unconsolidated Undrained
SAMPLE TYPE: Shelby
DESCRIPTION: Med.stiff,dk.gray
FAT CLAY(CH)-Trace shell frag

SPECIFIC GRAVITY= 2.7
REMARKS:

Fig. No.: ______

CLIENT: Geotechnical Consultants,Inc.

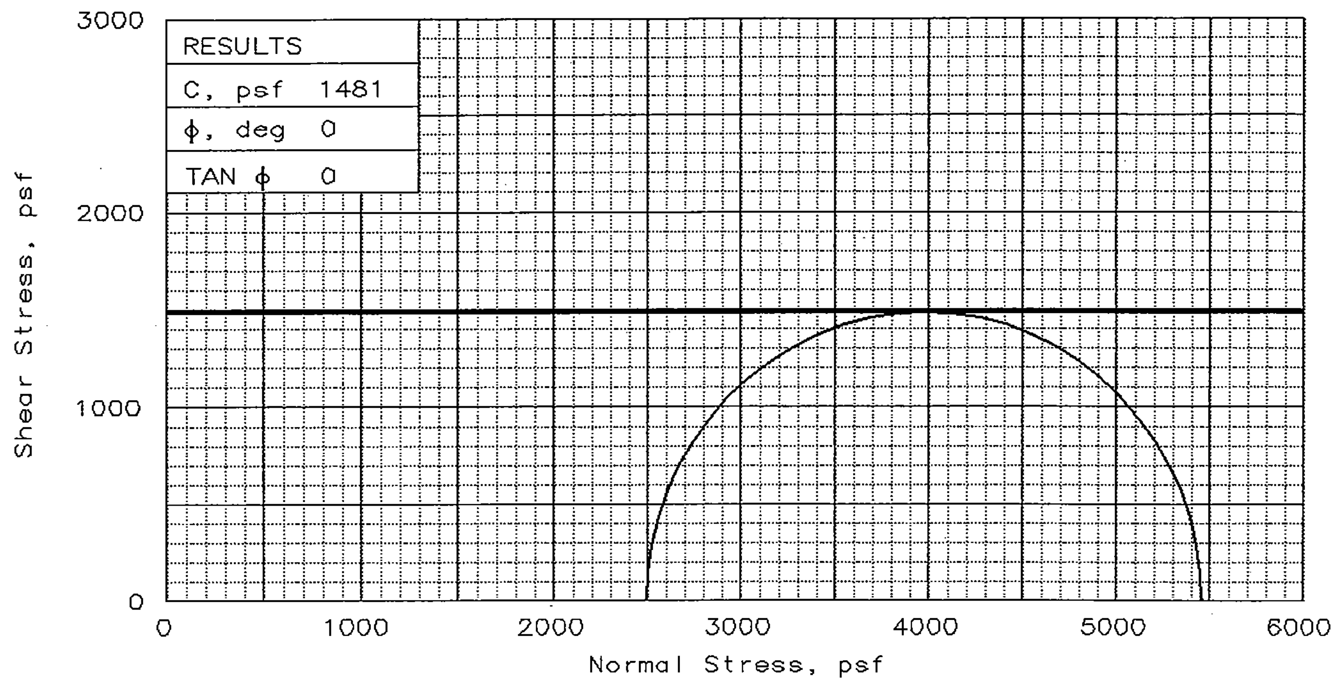
PROJECT: MUNI Power Plant

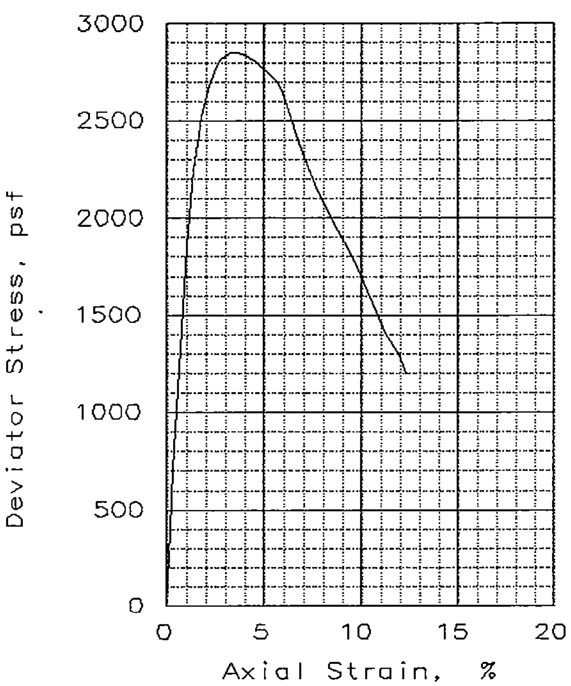
SAMPLE LOCATION: B-5 40-43'
Test @ 42.5'

PROJ. NO.: SF05019 DATE: 8-23-05

TRIAXIAL SHEAR TEST REPORT

Soil Mechanics Lab





SAMPLE NO.:		1
INITIAL	WATER CONTENT, %	52.3
	DRY DENSITY, pcf	68.0
	SATURATION, %	95.5
	VOID RATIO	1.478
	DIAMETER, in	2.88
	HEIGHT, in	5.68
AT TEST	WATER CONTENT, %	52.3
	DRY DENSITY, pcf	68.0
	SATURATION, %	95.5
	VOID RATIO	1.478
	DIAMETER, in	2.88
	HEIGHT, in	5.68
Strain rate, in/min		0.0750
BACK PRESSURE, psf		0
CELL PRESSURE, psf		2500
FAIL. STRESS, psf		2962
STRAIN, %		3.5
ULT. STRESS, psf		
STRAIN, %		
σ_1 FAILURE, psf		5462
σ_3 FAILURE, psf		2500

TYPE OF TEST:
Unconsolidated Undrained

SAMPLE TYPE: Shelby

DESCRIPTION: Stiff,dk.gray FAT CLAY(CH)-Trace shell frags.

SPECIFIC GRAVITY= 2.7

REMARKS:

Fig. No.: ______

CLIENT: Geotechnical Consultants,Inc.

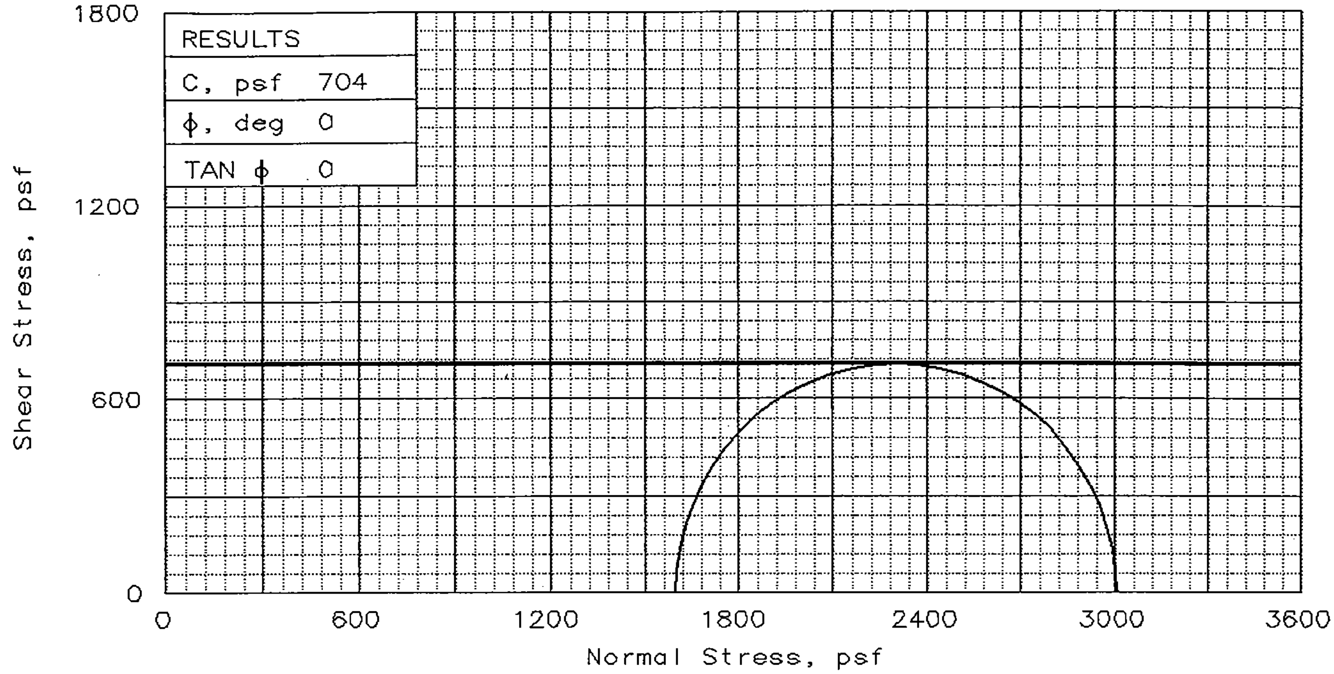
PROJECT: MUNI Power Plant

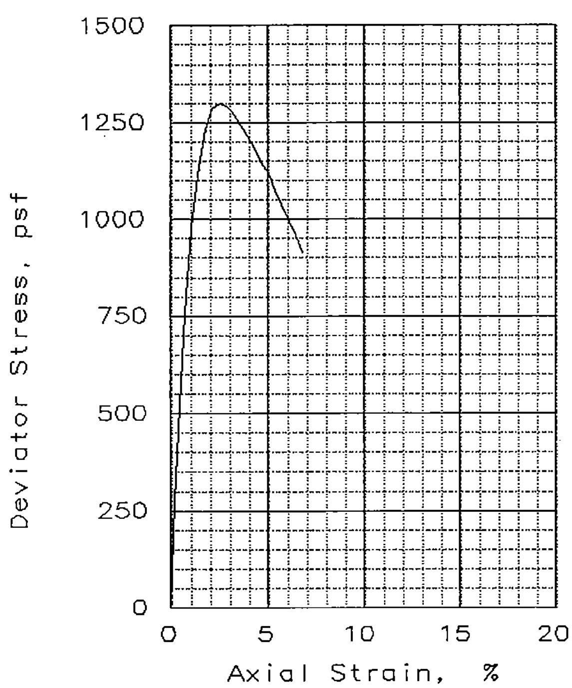
SAMPLE LOCATION: B-6 49-52'
Test @ 50.5'

PROJ. NO.: SF05019 DATE: 8-23-05

TRIAXIAL SHEAR TEST REPORT

Soil Mechanics Lab





SAMPLE NO.:		1
INITIAL	WATER CONTENT, %	51.5
	DRY DENSITY, pcf	69.6
	SATURATION, %	97.7
	VOID RATIO	1.423
	DIAMETER, in	2.88
	HEIGHT, in	5.57
AT TEST	WATER CONTENT, %	51.5
	DRY DENSITY, pcf	69.6
	SATURATION, %	97.7
	VOID RATIO	1.423
	DIAMETER, in	2.88
	HEIGHT, in	5.57
Strain rate, in/min		0.0750
BACK PRESSURE, psf		0
CELL PRESSURE, psf		1600
FAIL. STRESS, psf		1408
STRAIN, %		2.5
ULT. STRESS, psf		
STRAIN, %		
σ_1 FAILURE, psf		3008
σ_3 FAILURE, psf		1600

TYPE OF TEST:
Unconsolidated Undrained

SAMPLE TYPE: Shelby

DESCRIPTION: Med.stiff,dk.gray
FAT CLAY(CH)*

SPECIFIC GRAVITY= 2.7

REMARKS: With pockets of coarse
shell fragments

Fig. No.: ______

CLIENT: Geotechnical Consultants,Inc.

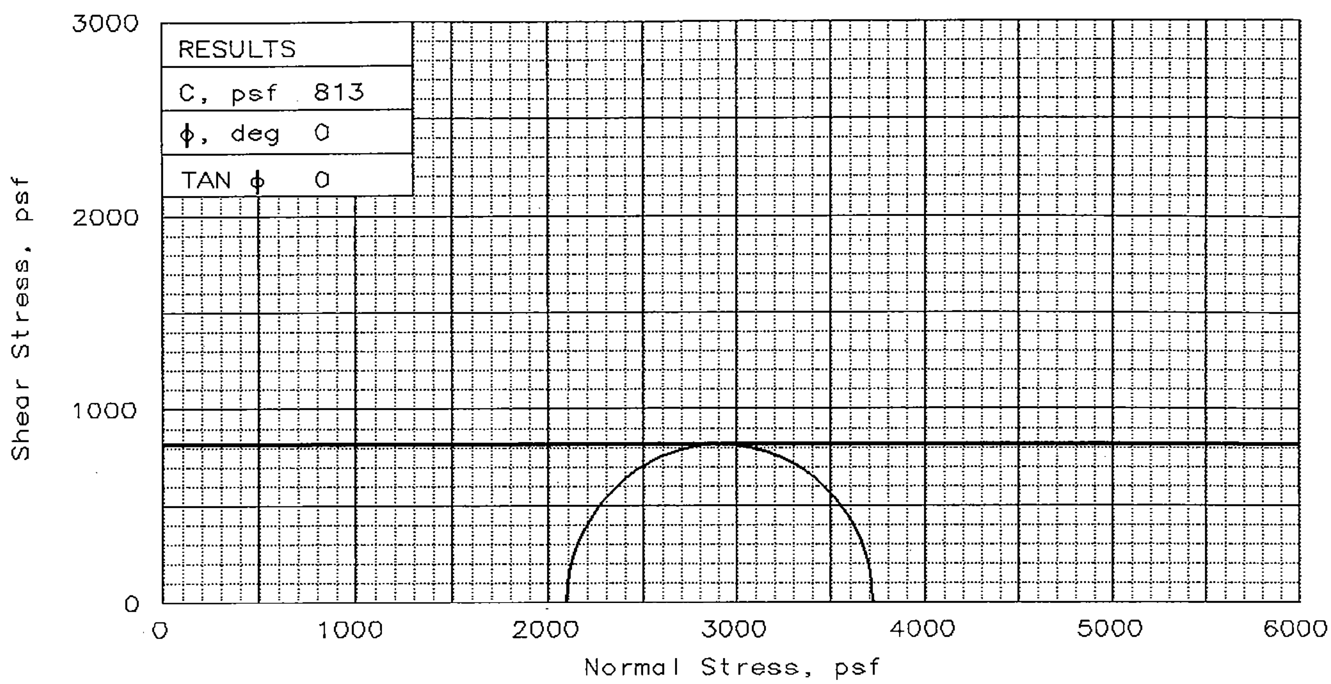
PROJECT: MUNI Power Plant

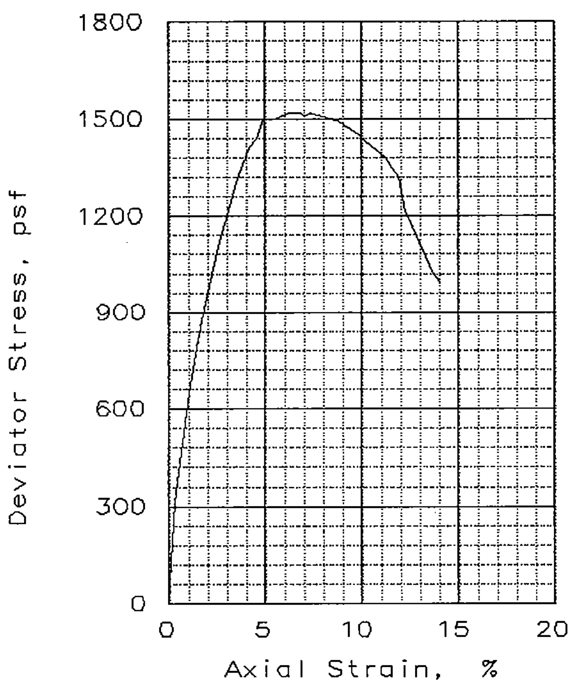
SAMPLE LOCATION: B-7 30-33'
Test @ 32'

PROJ. NO.: SF05019 DATE: 8-23-05

TRIAXIAL SHEAR TEST REPORT

Soil Mechanics Lab





SAMPLE NO.:		1
INITIAL	WATER CONTENT, %	54.4
	DRY DENSITY, pcf	67.4
	SATURATION, %	97.9
	VOID RATIO	1.501
	DIAMETER, in	2.88
	HEIGHT, in	5.70
AT TEST	WATER CONTENT, %	54.4
	DRY DENSITY, pcf	67.4
	SATURATION, %	97.9
	VOID RATIO	1.501
	DIAMETER, in	2.88
	HEIGHT, in	5.70
Strain rate, in/min		0.0750
BACK PRESSURE, psf		0
CELL PRESSURE, psf		2100
FAIL. STRESS, psf		1626
STRAIN, %		7.5
ULT. STRESS, psf		
STRAIN, %		
σ_1 FAILURE, psf		3726
σ_3 FAILURE, psf		2100

TYPE OF TEST:
Unconsolidated Undrained

SAMPLE TYPE: Shelby

DESCRIPTION: Med.stiff,dk.gray
CLAY(CH)

SPECIFIC GRAVITY= 2.7

REMARKS:

Fig. No.: ______

CLIENT: Geotechnical Consultants,Inc.

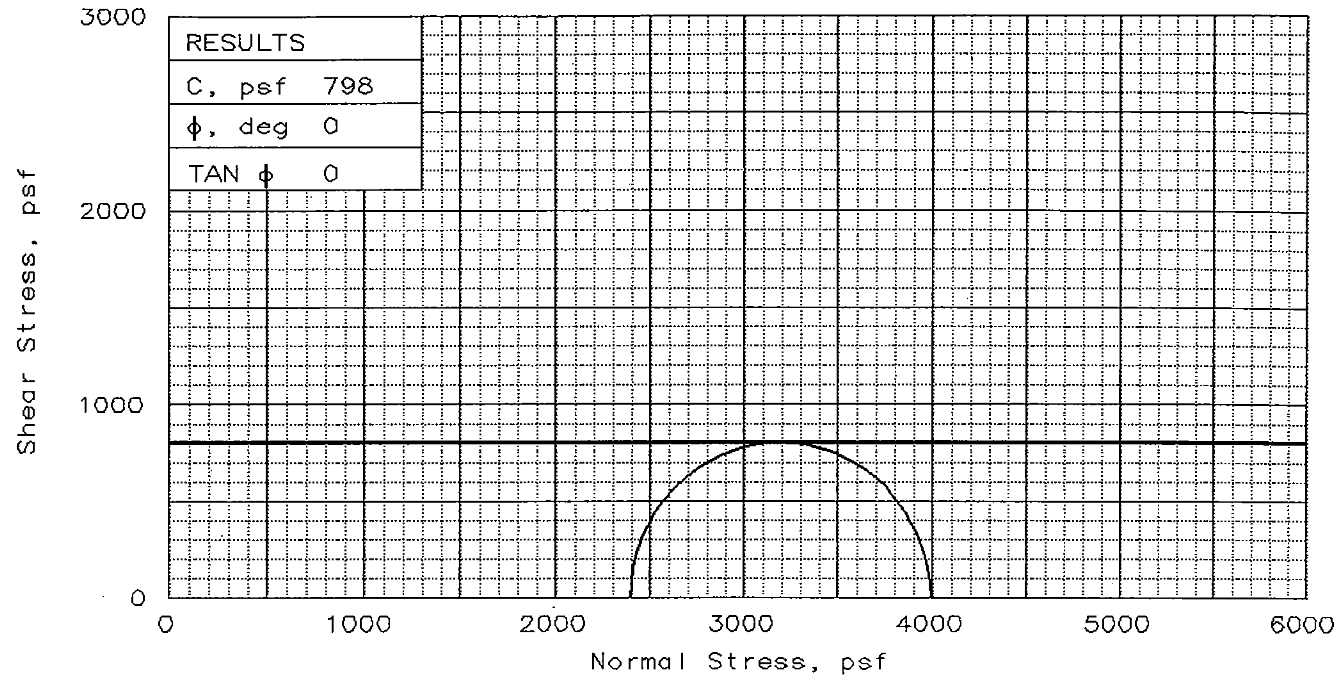
PROJECT: MUNI Power Plant

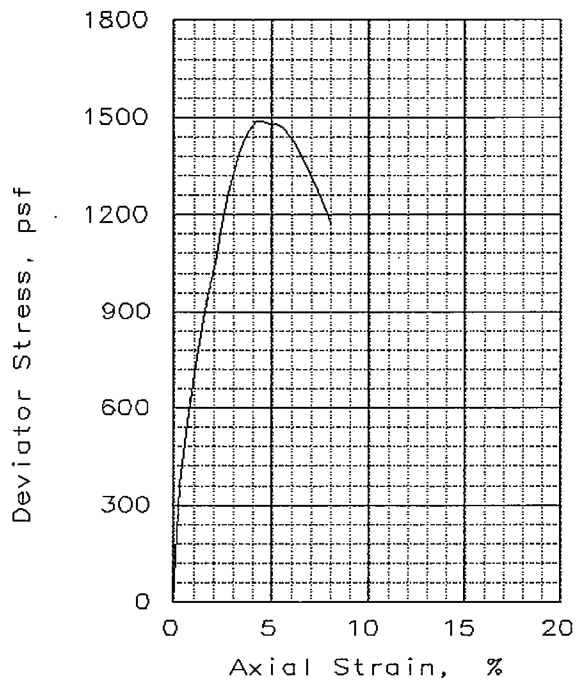
SAMPLE LOCATION: B-9 40-43'
Test @ 42'

PROJ. NO.: SF05019 DATE: 8-23-05

TRIAXIAL SHEAR TEST REPORT

Soil Mechanics Lab





SAMPLE NO.:		1
INITIAL	WATER CONTENT, %	50.6
	DRY DENSITY, pcf	68.4
	SATURATION, %	93.4
	VOID RATIO	1.463
	DIAMETER, in	2.88
	HEIGHT, in	5.70
AT TEST	WATER CONTENT, %	50.6
	DRY DENSITY, pcf	68.4
	SATURATION, %	93.4
	VOID RATIO	1.463
	DIAMETER, in	2.88
	HEIGHT, in	5.70

Strain rate, in/min	0.0750
BACK PRESSURE, psf	0
CELL PRESSURE, psf	2400
FAIL. STRESS, psf	1596
STRAIN, %	4.6
ULT. STRESS, psf	
STRAIN, %	
σ_1 FAILURE, psf	3996
σ_3 FAILURE, psf	2400

TYPE OF TEST:
Unconsolidated Undrained

SAMPLE TYPE: Shelby

DESCRIPTION: Soft, dk.gray FAT CLAY(CH)

SPECIFIC GRAVITY= 2.7

REMARKS:

Fig. No.: ______

CLIENT: Geotechnical Consultants, Inc.

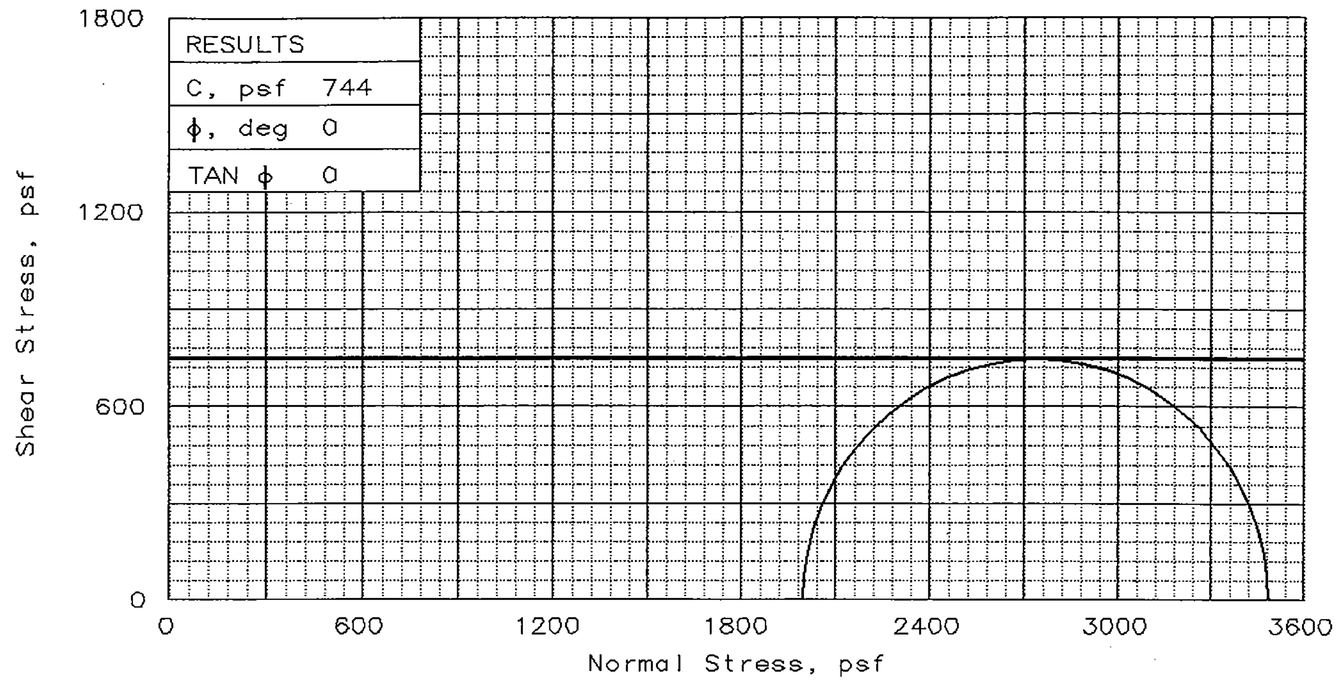
PROJECT: MUNI Power Plant

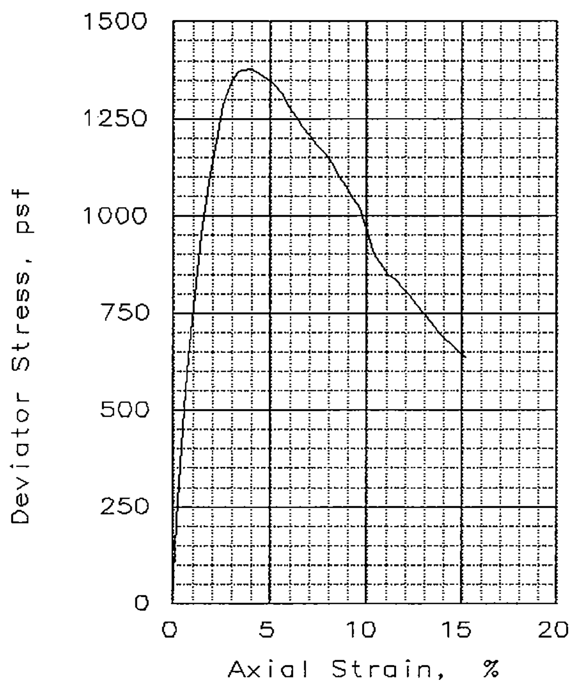
SAMPLE LOCATION: B-11 45-48'
Test @ 47.5'

PROJ. NO.: SF05019 DATE: 8-23-05

TRIAXIAL SHEAR TEST REPORT

Soil Mechanics Lab





SAMPLE NO.:		1
INITIAL	WATER CONTENT, %	55.2
	DRY DENSITY, pcf	66.8
	SATURATION, %	97.8
	VOID RATIO	1.524
	DIAMETER, in	2.88
	HEIGHT, in	5.39
AT TEST	WATER CONTENT, %	55.2
	DRY DENSITY, pcf	66.8
	SATURATION, %	97.8
	VOID RATIO	1.524
	DIAMETER, in	2.88
	HEIGHT, in	5.39
Strain rate, in/min		0.0750
BACK PRESSURE, psf		0
CELL PRESSURE, psf		2000
FAIL. STRESS, psf		1488
STRAIN, %		3.9
ULT. STRESS, psf		
STRAIN, %		
σ_1 FAILURE, psf		3488
σ_3 FAILURE, psf		2000

TYPE OF TEST:
Unconsolidated Undrained

SAMPLE TYPE: Shelby

DESCRIPTION: Med.stiff,dk.gray
FAT CLAY(CH)

SPECIFIC GRAVITY= 2.7

REMARKS:

Fig. No.: ______

CLIENT: Geotechnical Consultants,Inc.

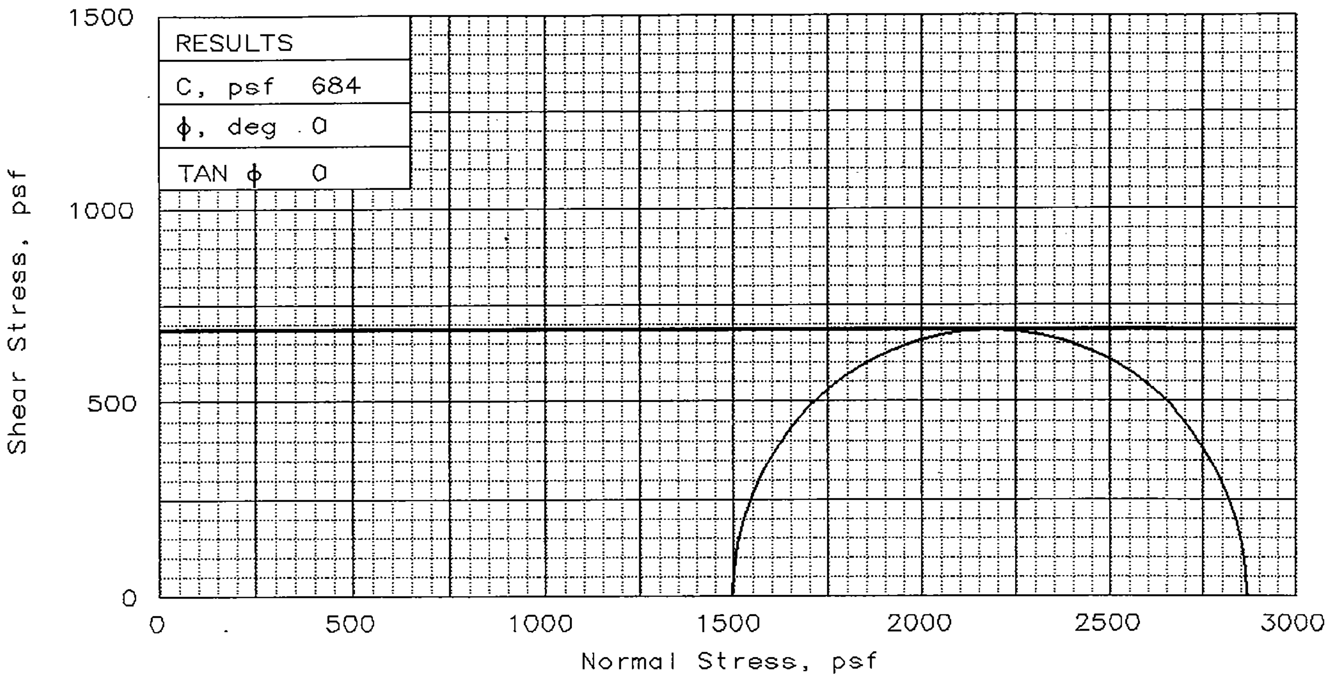
PROJECT: MUNI Power Plant

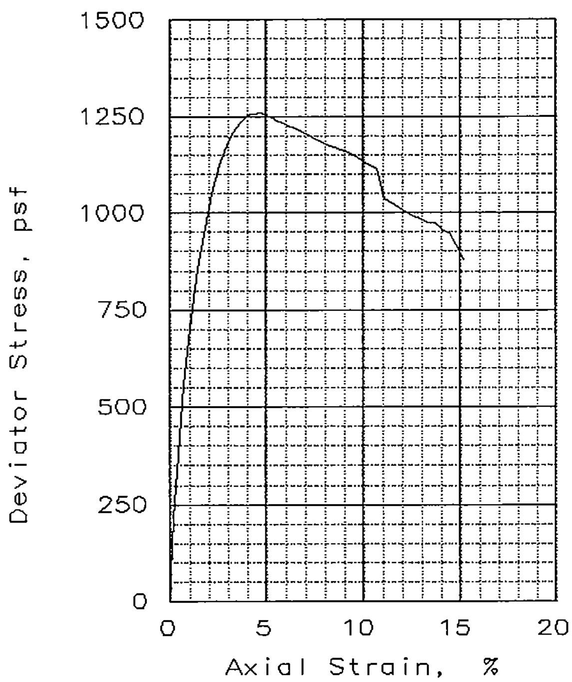
SAMPLE LOCATION: B-12 40-43'
Test @ 42.5'

PROJ. NO.: SF05019 DATE: 8-23-05

TRIAXIAL SHEAR TEST REPORT

Soil Mechanics Lab





SAMPLE NO.:		1
INITIAL	WATER CONTENT, %	56.3
	DRY DENSITY, pcf	65.8
	SATURATION, %	97.2
	VOID RATIO	1.563
	DIAMETER, in	2.88
	HEIGHT, in	5.40
AT TEST	WATER CONTENT, %	56.3
	DRY DENSITY, pcf	65.8
	SATURATION, %	97.2
	VOID RATIO	1.563
	DIAMETER, in	2.88
	HEIGHT, in	5.40
Strain rate, in/min		0.0750
BACK PRESSURE, psf		0
CELL PRESSURE, psf		1500
FAIL. STRESS, psf		1369
STRAIN, %		4.6
ULT. STRESS, psf		
STRAIN, %		
σ_1 FAILURE, psf		2869
σ_3 FAILURE, psf		1500

TYPE OF TEST:
Unconsolidated Undrained

SAMPLE TYPE: Shelby

DESCRIPTION: Med.stiff,dk.gray
FAT CLAY(CH)

SPECIFIC GRAVITY= 2.7

REMARKS:

Fig. No.: ______

CLIENT: Geotechnical Consultants,Inc.

PROJECT: MUNI Power Plant

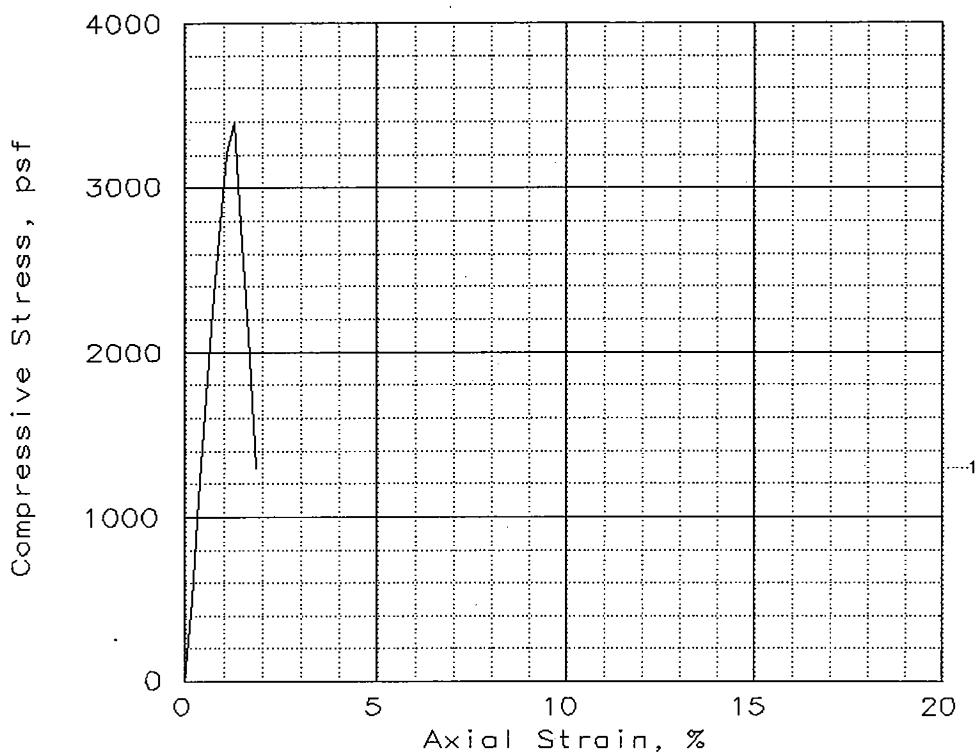
SAMPLE LOCATION: B-15 30-33'
Test @ 32.5'

PROJ. NO.: SF05019 DATE: 8-23-05

TRIAXIAL SHEAR TEST REPORT

Soil Mechanics Lab

UNCONFINED COMPRESSION TEST



SAMPLE NO.:	1			
Unconfined strength, psf	3394			
Undrained shear strength, psf	1697			
Failure strain, %	1.3			
Strain rate, in/min	0.0750			
Water content, % (cuttings before test)	57.3			
Wet density, pcf	106.7			
Dry density, pcf	67.8			
Saturation, %	104.2			
Void ratio	1.4854			
Specimen diameter, in	2.88			
Specimen height, in	5.40			
Height/diameter ratio	1.88			

1) Description: Stiff,brittle,gre- enish gray FAT CLAY(CH)

2) Description:

3) Description:

4) Description:

GS= 2.7 Type: Shelby

Project No.: SF05019

Date: 8-23-05

Remarks:

Fig. No.:

Client: Geotechnical Consultants,Inc.

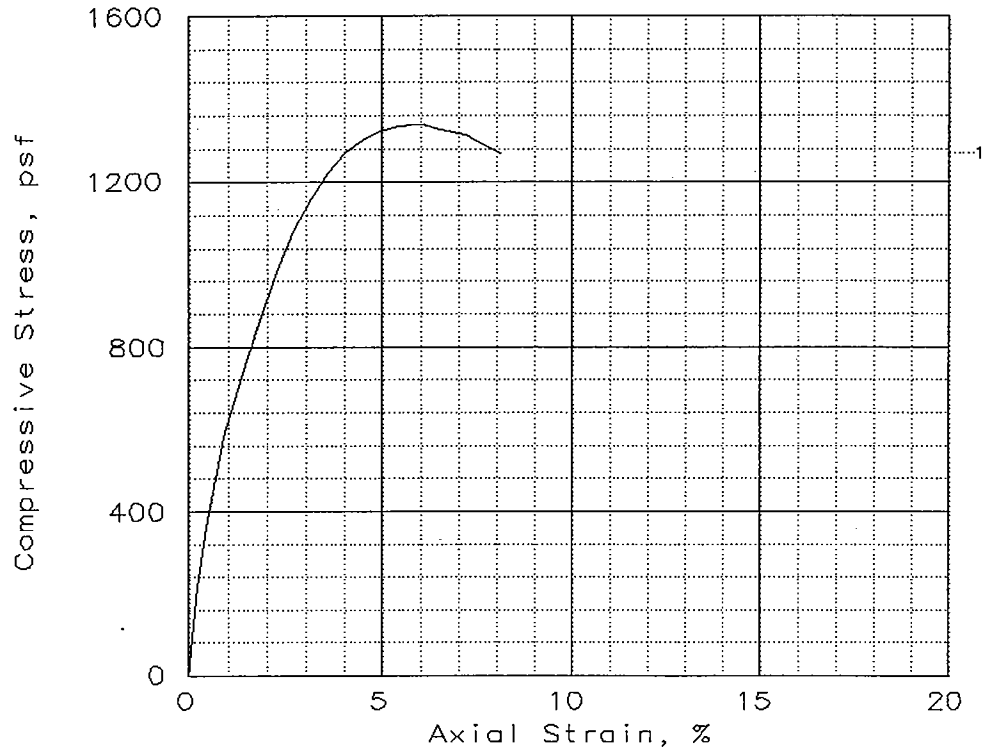
Project: MUNI Power Plant

Location: B-7 80-83'
Test @ 82.5'

UNCONFINED COMPRESSION TEST

Soil Mechanics Lab

UNCONFINED COMPRESSION TEST



SAMPLE NO.:	1			
Unconfined strength, psf	1339			
Undrained shear strength, psf	670			
Failure strain, %	5.9			
Strain rate, in/min	0.0750			
Water content, % (cuttings before test)	51.2			
Wet density, pcf	104.3			
Dry density, pcf	69.0			
Saturation, %	95.8			
Void ratio	1.4442			
Specimen diameter, in	2.42			
Specimen height, in	4.43			
Height/diameter ratio	1.83			

1) Description: Soft,dark gray FAT CLAY(CH)

2) Description:

3) Description:

4) Description:

			GS= 2.7	Type: MC

Project No.: SF05019

Date: 8-23-05

Remarks:

Client: Geotechnical Consultants,Inc.

Project: MUNI Power Plant

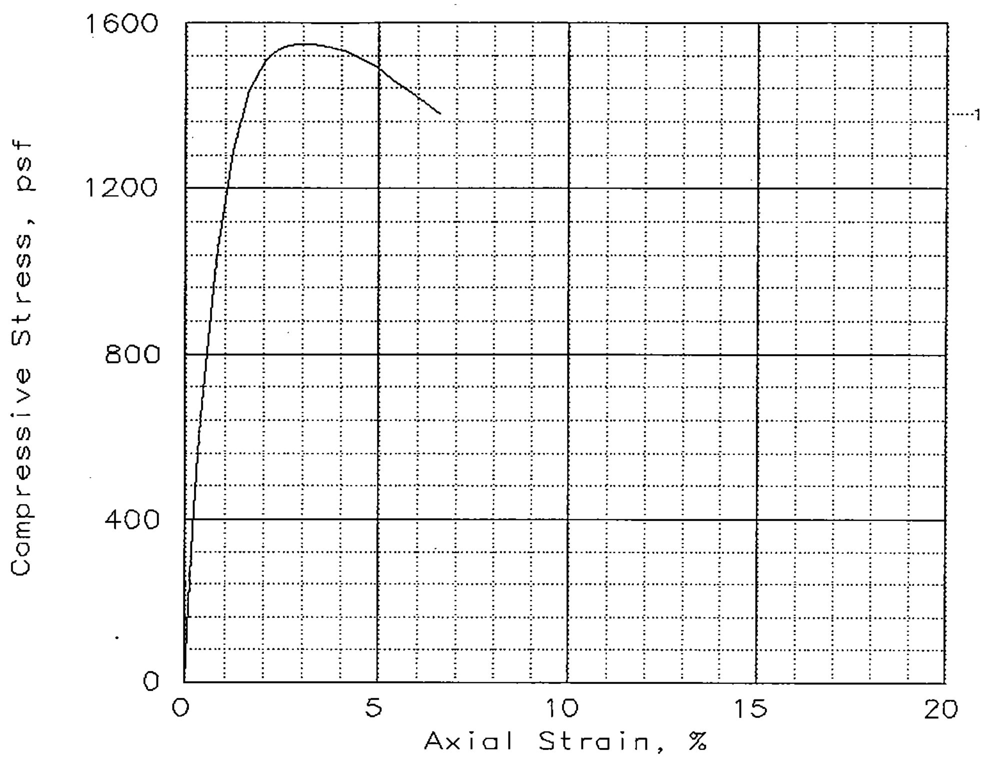
Location: B-9 90.5-91'

UNCONFINED COMPRESSION TEST

Soil Mechanics Lab

Fig. No.: ______

UNCONFINED COMPRESSION TEST



SAMPLE NO.:	1			
Unconfined strength, psf	1547			
Undrained shear strength, psf	773			
Failure strain, %	2.9			
Strain rate, in/min	0.0750			
Water content, % (cuttings before test)	50.9			
Wet density, pcf	105.5			
Dry density, pcf	69.9			
Saturation, %	97.3			
Void ratio	1.4116			
Specimen diameter, in	2.42			
Specimen height, in	4.85			
Height/diameter ratio	2.01			

1) Description: Med. stiff,dk.gray FAT CLAY(CH)

2) Description:

3) Description:

4) Description:

GS= 2.7 Type: MC

Project No.: SF05019
Date: 8-23-05
Remarks:

Client: Geotechnical Consultants,Inc.

Project: MUNI Power Plant

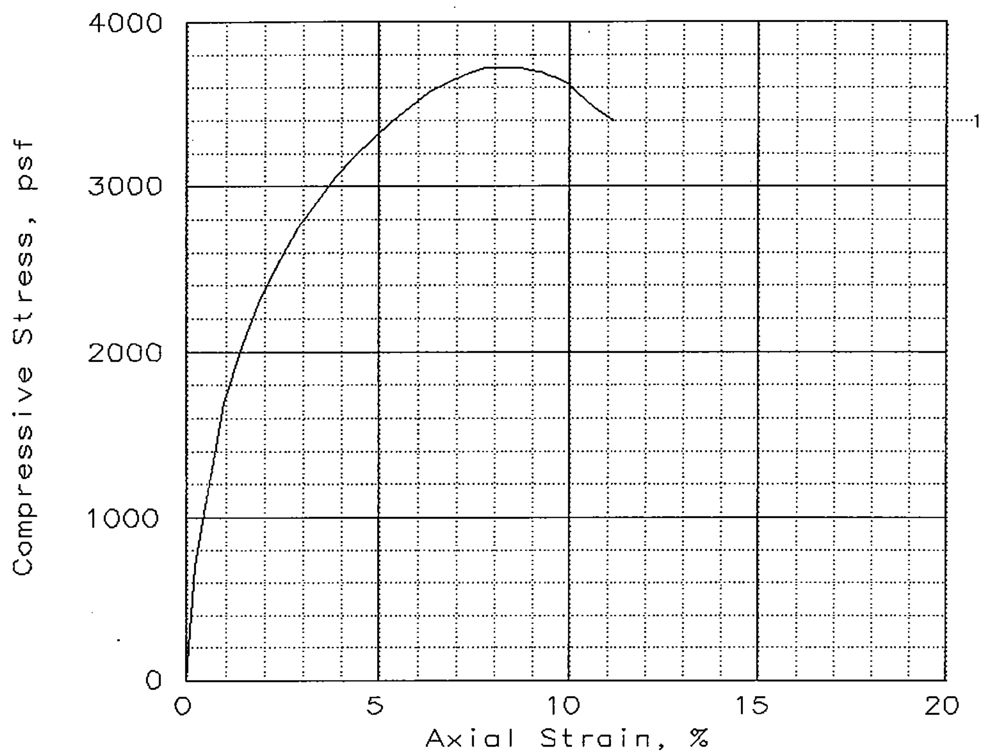
Location: B-11 90.5-91'

UNCONFINED COMPRESSION TEST

Soil Mechanics Lab

Fig. No.: ____

UNCONFINED COMPRESSION TEST



SAMPLE NO.:	1			
Unconfined strength, psf	3722			
Undrained shear strength, psf	1861			
Failure strain, %	8.5			
Strain rate, in/min	0.0750			
Water content, % (cuttings before test)	27.6			
Wet density, pcf	122.3			
Dry density, pcf	95.9			
Saturation, %	98.1			
Void ratio	0.7583			
Specimen diameter, in	2.42			
Specimen height, in	4.12			
Height/diameter ratio	1.71			

1) Description: Med. stiff,dark bluish gray FAT CLAY(CH)

2) Description:

3) Description:

4) Description:

GS= 2.7 Type: MC

Project No.: SF05019

Date: 8-23-05

Remarks:

Fig. No.: ____

Client: Geotechnical Consultants,Inc.

Project: MUNI Power Plant

Location: B-15 90-90.5'

UNCONFINED COMPRESSION TEST

Soil Mechanics Lab

APPENDIX B –
SITE SPECIFIC DYNAMIC RESPONSE ANALYSIS

Robert Pyke, Consulting Engineer

September 11, 2005

Amy Killeen
Geotechnical Consultants, Inc.
500 Sansome Street, Suite 402
San Francisco CA 94111

Re: SFPUC - "Muni" Power Plant Site

Dear Amy,

At your request I have conducted seismic hazard analyses for horizontal ground motions at this site with a probability of exceedance of 10 percent in 50 years and 2 percent in 50 years with a view to constructing design response spectra in accordance with FEMA 356 using the site-specific procedure detailed in Section 1.6.2.

The site is located near the Islais Creek Channel on the southern San Francisco waterfront and is approximately 12 km from the San Andreas fault and 17 km from the Hayward fault. Your geotechnical investigation has revealed that the site has a strongly layered soil profile with the surface fill materials being underlain by colluvium, then young Bay Mud, then alluvial deposits that are sometimes known as the Upper Layered Sediments, then Old Bay Clay, then sediments that are sometimes known as the Lower Layered Sediments and finally Franciscan bedrock. The exact shear wave velocity of the bedrock at this site is not known but it can be assumed that the standard ground motion attenuation relationships for soft rock are applicable.

In order to obtain probabilistic response spectra I have conducted a formal probabilistic seismic hazard analysis using the hazard analysis procedure that was originally suggested by Cornell (1968) and is embodied in the computer program EQRISK, as described by McGuire (1976). The locations of the source zones and the assumed source zone parameters that were used are based on data presented by Petersen et al. (1996) and USGS (1999,2003).

Since EQRISK models only areal sources rather than line sources, fault zones are normally modelled as strips having widths of about 2 km. However, in this case, because ground motion at the site will be controlled by larger events on the San Andreas and Hayward faults for which fault rupture must pass opposite the site regardless of the point of initiation and the length of the rupture, these zones were truncated to shorter lengths in order to force use of appropriate distances in computing spectral accelerations.

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Further, since EQRISK otherwise assumes that the occurrence of earthquakes is randomly distributed in time, the activities assigned to the zones representing the larger earthquakes have been adjusted using a procedure suggested by Cornell and Winterstein (1988) in order to account for the date of last occurrence of major earthquakes on the San Andreas and Hayward fault systems, assuming a window of exposure of 50 years. For the Hayward fault it has been assumed that the Northern and Southern segments can rupture independently and the activity of the Northern segment has been increased to take into account the present uncertainty regarding the date of last rupture. The maximum magnitude on both the North and South segments of the Hayward fault has been taken to be 7.0. This is higher than the values assigned by the 2002 USGS Working Group (WG02) but allows for the possibility that the two segments might rupture in a single event. The possibility that smaller events will occur away from major mapped faults or on minor faults is accounted for by the inclusion of "background" activity. The assumed source zone parameters are listed in Table 1, in which M indicates moment magnitude and b is the Richter and Gutenberg b parameter.

The attenuation relationships for 5 percent damped spectral acceleration on rock sites developed by Abrahamson & Silva (1997) and Sadigh et al. (1997) were used in the analyses. These relationships are currently being updated in a study co-ordinated by the Pacific Earthquake Engineering Research Center (PEER) that is referred to as the Next Generation Attenuation (NGA) study. Formal results from the NGA study are not yet available but preliminary results suggest that the new relationships may show lower spectral accelerations for periods less than 1 second particularly for faults like the San Andreas and Hayward faults that exhibit surface rupture and have a high aspect ratio (the ratio of length to width (that is, the depth of the fault rupture)). Should the short period motions be critical to the project it may be possible to reduce them once the NGA study is completed but the date of the release of the formal results is presently uncertain (Maury Power, personal communication, July 18). Pending completion of the NGA study I have elected not to explicitly address forward directivity effects in my analyses. While some workers believe that these can be significant, work conducted for the New East Spans of the Bay Bridge in which I participated suggested that there is some uncertainty regarding these effects and I believe that it is adequately accommodated by the uncertainties that are already included in the analyses.

The 5 percent damped horizontal response spectra obtained for probabilities of exceedance of 10 percent in 50 years and 2 percent in 50 years are shown in the attached Figures 1 and 2. The values obtained are quite large as a result of the proximity of the site to the San Andreas fault however FEMA 356 provides for alternate use of spectra based on a deterministic evaluation of ground motion as follows:

For BSE-2 the acceleration parameters used to construct the design spectrum are taken as the **smaller** of the values derived from:

(1) the 2 percent in 50 years spectrum; and
(2) 150 percent of the mean deterministic spectrum.

For BSE-1 the acceleration parameters used to construct the design spectrum are taken as the **smaller** of the values derived from:

(1) the 10 percent in 50 years spectrum; and
(2) two-thirds of the BSE-2 spectrum (which is identical to the mean deterministic spectrum when BSE-2 is governed by the deterministic evaluation).

In evaluating the mean deterministic spectrum I have used a magnitude 7.8 earthquake at a distance of 12 km to represent the motion at the site generated by rupture of the San Andreas fault.

The resulting response spectra are shown in Figure 1 and the same spectra multiplied by 1.5 are shown in Figure 2. For comparison the mean plus one standard deviation spectra, which are often used in local practice to represent the MCE, are shown in Figure 3. These are marginally higher but not dissimilar to the mean deterministic spectra multiplied by 1.5. It is clear from these figures that the deterministic spectra should control the design in both cases.

In order to obtain horizontal motions at the ground surface one-dimensional nonlinear site response analyses were conducted. Such analyses require acceleration or velocity histories as input and therefore the first step in these analyses was to develop appropriate input acceleration histories. This was accomplished by fitting the two horizontal components of each of two motions used to represent an earthquake on the San Andreas fault that were originally developed by Dr Norman Abrahamson for use in the design of the new East Spans of the San Francisco Oakland Bay Bridge to the mean and 1.5 times mean deterministic spectra using the frequency domain fitting procedure that is included in the computer program TINKER (Tagasoft, 2004). Because it extends out to a period of 5 seconds and is defined by more points the Abrahamson and Silva spectra were used. Prior to fitting the acceleration histories the target spectra were extended to 10 seconds at constant displacement.

In conventional "equivalent linear" analyses of site response it is necessary to specify the shear wave velocity, or the shear modulus at small strains, G_{max}, for each layer along with modulus reduction and damping curves for each layer or material type. Modulus reduction curves of this kind can also be used as the "backbone" curve for constructing simple nonlinear models of shear stress - shear strain behavior. Pyke et al. (1993) constructed a consistent family of modulus reduction and damping curves in terms of the Hardin and Drnevich reference strain (t_{max}/G_{max}) which also accounts for rate of strain effects on small strain damping. The appropriate reference strains and other parameters required to model the soils at the SFIA site have been assigned on the basis of published data (especially EPRI, 1993) and our experience analyzing similar profiles. The degradation of the stiffnesses of Bay Mud layers was modelled using the scheme suggested by Idriss et al (1978). Because only limited excess pore pressure development and softening are anticipated in the Layered Sediments, the same degradation parameters were also applied to this layer. The shear wave velocity profile adopted for analysis was based on a combination of data from other sites and the values measured to a depth of 100 feet between Borings B10 and B11 by SouthWest Geophysics, Inc.

The nonlinear site response analyses were conducted using the computer program TESS (TAGAsoft, 2004). TESS employs an explicit finite difference solution of the one-dimension wave propagation problem. If the material property option to generate excess pore pressures is exercised, then optionally, a parallel solution of the one-dimensional diffusion problem can be run to re-distribute and dissipate excess pore pressures but that was not done in this case.

Examples of the printed output of the TESS runs are attached. The analyses for both the BSE-1 and BSE-2 levels of loading showed pronounced nonlinearity as a result of the fill serving as an inertial reaction that generates large shear strains in the young Bay Mud. As a result the ground surface motions that are obtained for BSE-2 are not much greater than those for BSE-1.

The maximum displacements that are shown in the printed output do not necessarily occur at the same times but can be used to guide the detailing of piles to provide necessary ductility. Should it be desired to conduct more detailed analyses of pile bending, TESS could be re-run in order to save as many free-field displacement histories as desired. While in some cases it would be standard practice to test the sensitivity of the results to variations in the assumed profile and properties, because the pronounced nonlinearity dominates the results that has not been done at this time.

The computed ground surface motions were saved and were used to compute 2, 5 and 10 percent damped spectra, as shown in Figures 4-6 for the BSE-1 motions and Figures 7-9 for the BSE-2 motions. The computed spectra were then averaged in order to obtain the spectra that are recommended for design and that are shown in Figure 10 for BSE-1 and Figure 11 for BSE-2. Because the pronounced nonlinearity of the response leads to non-standard response spectra shapes it is not possible to use the construction for developing design response spectra that is described in FEMA 356.

Should vertical response spectra be required FEMA 356 requires that it be taken to have two-thirds of the spectral accelerations of the horizontal spectra, however, should vertical motions be critical I would recommend that, based on results obtained using the relationships for horizontal and vertical motions of Abrahamson and Silva (1997), the vertical spectra should be taken to be equal to the horizontal spectra up to a period of 0.15 seconds, drop linearly to be equal to one-half of the horizontal spectra at a period of 0.5 seconds, and remain at one-half the horizontal spectra at longer periods.

Please contact me should you or the project structural engineer have any questions.

Sincerely,

Robert Pyke

Robert Pyke, Ph.D., G.E.

REGISTERED PROFESSIONAL ENGINEER
ROBERT PYKE
GE00702
Exp. 03-31-05
GEOTECHNICAL
STATE OF CALIFORNIA

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Table 1
Source Parameters

	GROSS SOURCE	Mmin	Mmax	b	EVENTS/YR
1	Northern San Andreas	8.00	8.25	.1000	.0020
2	Peninsula S.A. Large	6.50	7.00	.1000	.0080
3	Peninsula S.A. Small	4.00	6.50	.9000	.1300
4	Rogers Creek Large	6.50	7.00	.1000	.0070
5	Rogers Creek Small	4.00	6.50	.9000	.2000
6	Hayward Large North	6.50	7.00	.1000	.0110
7	Hayward Large South	6.50	7.00	.1000	.0080
8	Hayward Small	4.00	6.50	.9000	.1300
9	Concord	4.00	6.80	.9000	.0500
10	Mount Diablo Thrust	6.50	7.00	.1000	.0020
11	Calaveras	4.00	6.50	.9000	.1000
12	Greenville	4.00	6.50	.9000	.0500
13	Background	4.00	5.50	.9000	.3000

Table 2
Ground Surface Spectra for BSE-1

10 Percent Damping

PERIOD IN SECONDS	SPECTRAL ACCELERATION (G)
0.01	0.205
0.1	0.26
0.2	0.32
0.3	0.39
0.4	0.43
0.5	0.48
0.6	0.50
0.7	0.52
0.8	0.54
0.9	0.5425
1.0	0.54
1.1	0.53
1.2	0.52
1.3	0.50
1.4	0.47
1.5	0.44
1.6	0.42
1.8	0.36
2.0	0.30
2.2	0.26
2.4	0.21
2.6	0.18
2.8	0.155
3.0	0.13

5 Percent Damping

PERIOD IN SECONDS	SPECTRAL ACCELERATION (G)
0.01	0.205
0.1	0.30
0.2	0.40
0.3	0.52
0.4	0.61
0.5	0.68
0.6	0.70
0.7	0.72
0.8	0.72
0.9	0.70
1.0	0.69
1.1	0.67
1.2	0.65
1.3	0.62
1.4	0.58
1.5	0.55
1.6	0.52
1.8	0.45
2.0	0.38
2.2	0.32
2.4	0.25
2.6	0.21
2.8	0.18
3.0	0.16

2 Percent Damping

PERIOD IN SECONDS	SPECTRAL ACCELERATION (G)
0.01	0.205
0.1	0.40
0.2	0.58
0.3	0.75
0.4	0.92
0.5	1.00
0.6	1.02
0.7	1.03
0.8	1.03
0.9	1.01
1.0	1.00
1.1	0.97
1.2	0.94
1.3	0.90
1.4	0.87
1.5	0.81
1.6	0.75
1.8	0.65
2.0	0.55
2.2	0.45
2.4	0.36
2.6	0.3
2.8	0.23
3.0	0.18

Table 3
Ground Surface Spectra for BSE-2

10 Percent Damping

PERIOD IN SECONDS	SPECTRAL ACCELERATION (G)
0.01	0.26
0.1	0.31
0.2	0.38
0.3	0.45
0.4	0.50
0.5	0.54
0.6	0.57
0.7	0.60
0.8	0.62
0.9	0.64
1.0	0.65
1.1	0.64
1.2	0.63
1.3	0.62
1.4	0.60
1.5	0.58
1.6	0.55
1.8	0.50
2.0	0.45
2.2	0.40
2.4	0.33
2.6	0.29
2.8	0.25
3.0	0.21

5 Percent Damping

PERIOD IN SECONDS	SPECTRAL ACCELERATION (G)
0.01	0.26
0.1	0.38
0.2	0.51
0.3	0.62
0.4	0.73
0.5	0.78
0.6	0.82
0.7	0.85
0.8	0.87
0.9	0.8725
1.0	0.87
1.1	0.86
1.2	0.83
1.3	0.79
1.4	0.76
1.5	0.74
1.6	0.70
1.8	0.64
2.0	0.55
2.2	0.48
2.4	0.41
2.6	0.35
2.8	0.31
3.0	0.27

2 Percent Damping

PERIOD IN SECONDS	SPECTRAL ACCELERATION (G)
0.01	0.26
0.1	0.47
0.2	0.68
0.3	0.87
0.4	1.04
0.5	1.15
0.6	1.20
0.7	1.25
0.8	1.27
0.9	1.26
1.0	1.25
1.1	1.23
1.2	1.19
1.3	1.15
1.4	1.11
1.5	1.07
1.6	1.02
1.8	0.90
2.0	0.77
2.2	0.69
2.4	0.60
2.6	0.51
2.8	0.42
3.0	0.34

Example Outputs
For Nonlinear Site Response Analyses

The following pages show the printed output from the computer program TESS for the runs using the SA3n input motion for both the BSE1 and BSE2 levels of ground motion.

Definitions of key column headings are as follows:

SIGV - vertical effective stress
VS - shear wave velocity
GMAX - shear modulus at low strains
SHEAR STRENGTH - asymptote of stress-strain curve under rapid loading
REFERENCE STRAIN - ratio of shear strength to shear modulus at low strains

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3	2	10.00	.120			2.06	950.00	3363.35	3.363	.100	.000	.950
4	1	10.00	.105			2.57	550.00	986.41	2.959	.300	.000	.550
5	1	10.00	.102			2.98	500.00	791.93	2.376	.300	.000	.500
6	1	10.00	.100			3.36	450.00	628.88	1.887	.300	.000	.450
7	1	10.00	.120			3.84	900.00	3018.63	6.037	.200	.000	.900
8	1	10.00	.125			4.44	950.00	3503.49	7.007	.200	.000	.950
9	1	10.00	.110			4.99	500.00	854.04	2.562	.300	.000	.500
10	1	10.00	.110			5.47	525.00	941.58	2.825	.300	.000	.525
11	1	10.00	.110			5.94	550.00	1033.39	3.100	.300	.000	.550
12	1	10.00	.110			6.42	575.00	1129.46	3.388	.300	.000	.575
13	1	10.00	.110			6.89	600.00	1229.81	3.689	.300	.000	.600
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15	1	10.00	.125			7.92	940.00	3430.12	6.860	.200	.000	.940
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UNIT WEIGHT OF BASE = .125

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WITH A PEAK ACCELERATION OF .35 G
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7	60.00	.259	1.269	.472	26.506	.260	1.155	.051	.963	.963	.963	.000	.000	65.00
8	70.00	.252	1.259	.465	26.506	.256	1.293	.048	.965	.965	.965	.000	.000	75.00
9	80.00	.250	1.249	.459	26.496	.253	1.555	.457	.871	.871	.871	.000	.000	85.00
10	90.00	.312	1.215	.358	26.506	.197	1.721	.441	.878	.878	.878	.000	.000	95.00
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12	110.00	.298	1.316	.198	26.476	.107	1.941	.390	.895	.895	.895	.000	.000	115.00
13	120.00	.348	1.327	.136	26.466	.062	2.115	.362	.901	.901	.901	.000	.000	125.00
14	130.00	.332	1.307	.077	26.526	.026	2.275	.324	.906	.906	.906	.000	.000	135.00
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UNITS ARE KIPS, FEET AND SECONDS


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MATERIAL PROPERTY PARAMETERS

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  2         .02       .02      1.00       .00       .00       .00


PARAMETERS FOR SIMPLE DEGRADATION

   MTYPE   SS        RS        E         SG        RG        ST        RT
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LAYER DATA

TRAVEL TIMES ARE RELATIVE TO A TIMESTEP OF .010 SECONDS

LAYER NO.	MTYPE	THICK	UNIT WT	OCR	KO	SIGV	VS	GMAX	TAUMAX	GAMREF	TAUI	TTR
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3	2	10.00	.120			2.06	950.00	3363.35	3.363	.100	.000	.950
4	1	10.00	.105			2.57	550.00	986.41	2.959	.300	.000	.550
5	1	10.00	.102			2.98	500.00	791.93	2.376	.300	.000	.500
6	1	10.00	.100			3.36	450.00	628.88	1.887	.300	.000	.450
7	1	10.00	.120			3.84	900.00	3018.63	6.037	.200	.000	.900
8	1	10.00	.125			4.44	950.00	3503.49	7.007	.200	.000	.950
9	1	10.00	.110			4.99	500.00	854.04	2.562	.300	.000	.500
10	1	10.00	.110			5.47	525.00	941.58	2.825	.300	.000	.525
11	1	10.00	.110			5.94	550.00	1033.39	3.100	.300	.000	.550
12	1	10.00	.110			6.42	575.00	1129.46	3.388	.300	.000	.575
13	1	10.00	.110			6.89	600.00	1229.81	3.689	.300	.000	.600
14	1	10.00	.110			7.37	625.00	1334.43	4.003	.300	.000	.625
15	1	10.00	.125			7.92	940.00	3430.12	6.860	.200	.000	.940
16	1	10.00	.125			8.55	960.00	3577.64	7.155	.200	.000	.960
17	1	10.00	.125			9.17	960.00	3577.64	7.155	.200	.000	.960

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4	30.00	.253	2.131	.856	26.606	.584	.906	.160	.916	.916	.916	.000	.000	35.00
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8	70.00	.409	1.887	.675	26.626	.461	1.764	.068	.958	.958	.958	.000	.000	75.00
9	80.00	.326	1.869	.665	26.626	.454	2.038	.721	.829	.829	.829	.000	.000	85.00
10	90.00	.481	1.873	.498	26.616	.320	2.250	.646	.845	.845	.845	.000	.000	95.00
11	100.00	.513	1.891	.386	26.606	.227	2.391	.580	.858	.858	.858	.000	.000	105.00
12	110.00	.439	1.951	.302	26.596	.170	2.658	.484	.871	.871	.871	.000	.000	115.00
13	120.00	.457	1.966	.202	26.586	.116	2.900	.461	.877	.877	.877	.000	.000	125.00
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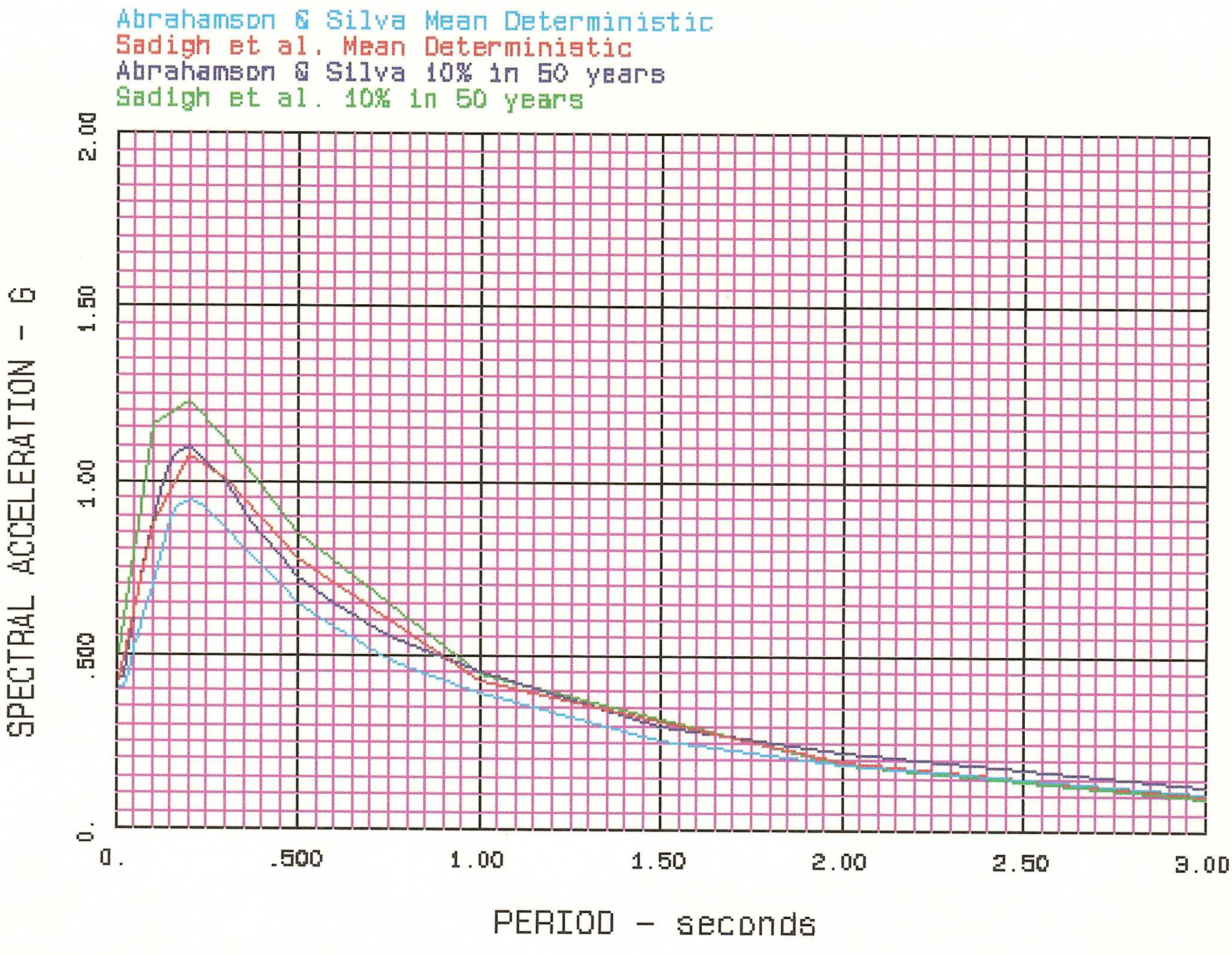

Fig.1 PUC POWER PLANT MUNI SITE
10% IN 50 YEARS + MEAN DETERMINISTIC

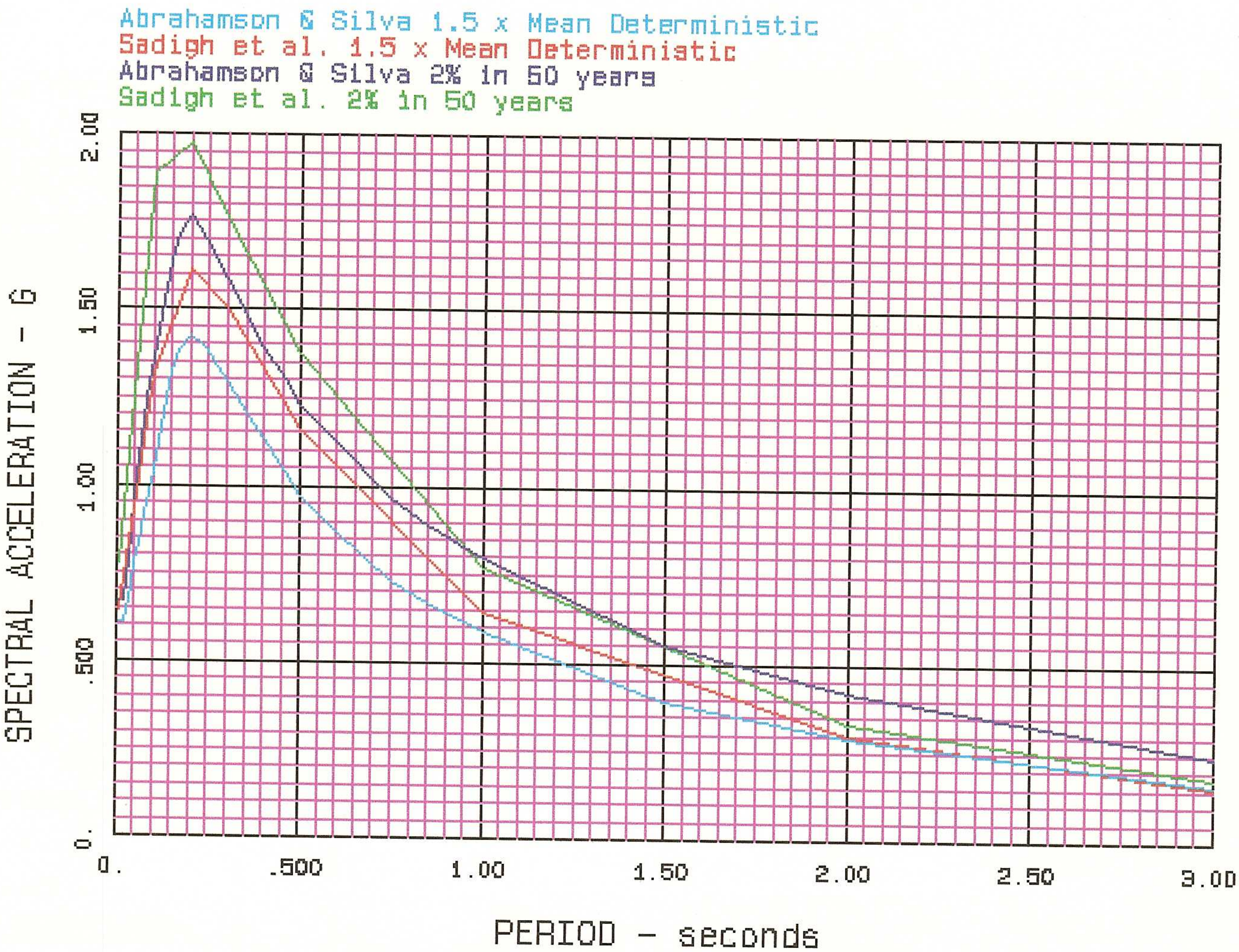

Fig.2 PUC POWER PLANT MUNI SITE
2% IN 50 YRS + 1.5 x MEAN DETERMINISTIC

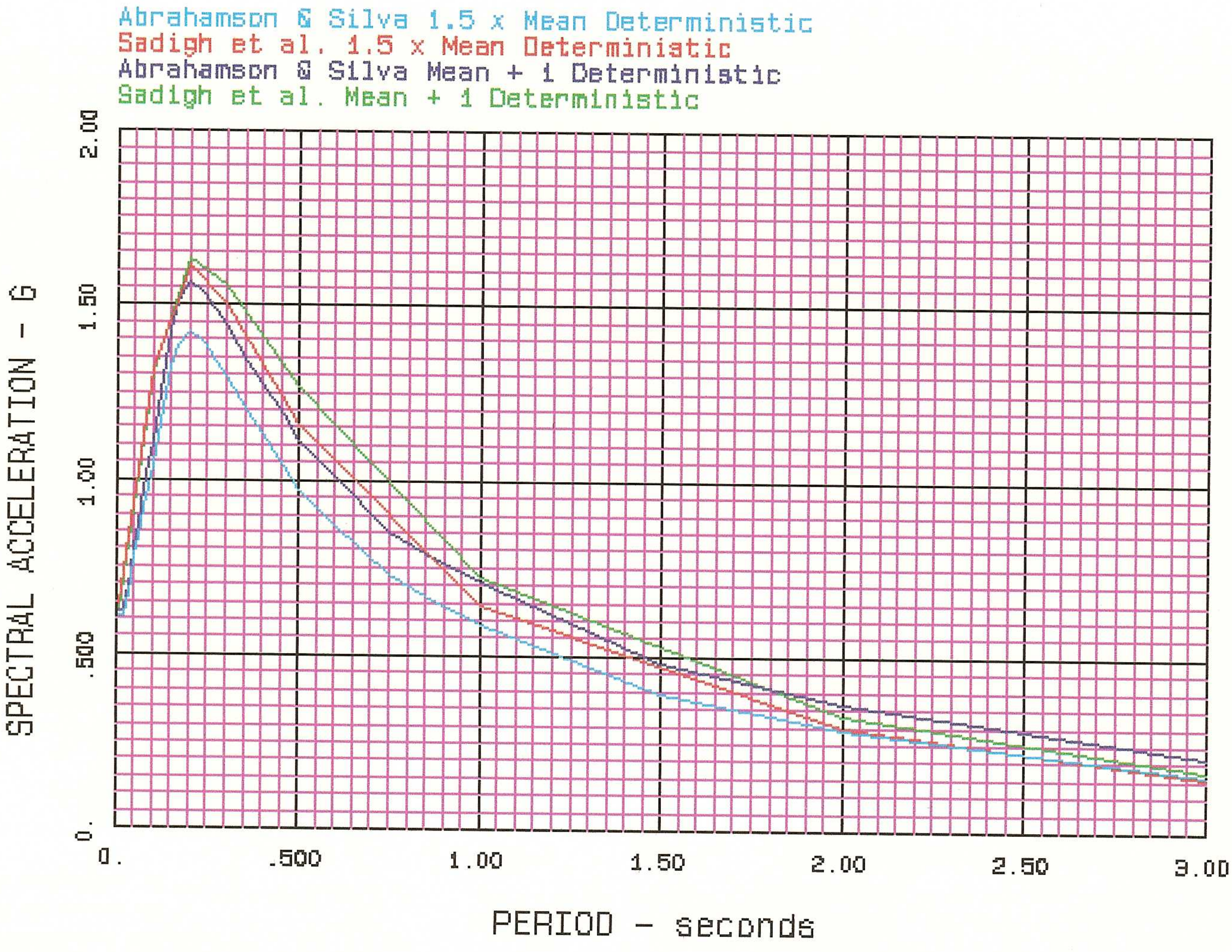

Fig.3 PUC POWER PLANT MUNI SITE
1.5 x MEAN DETERMINISTIC v. MEAN + 1

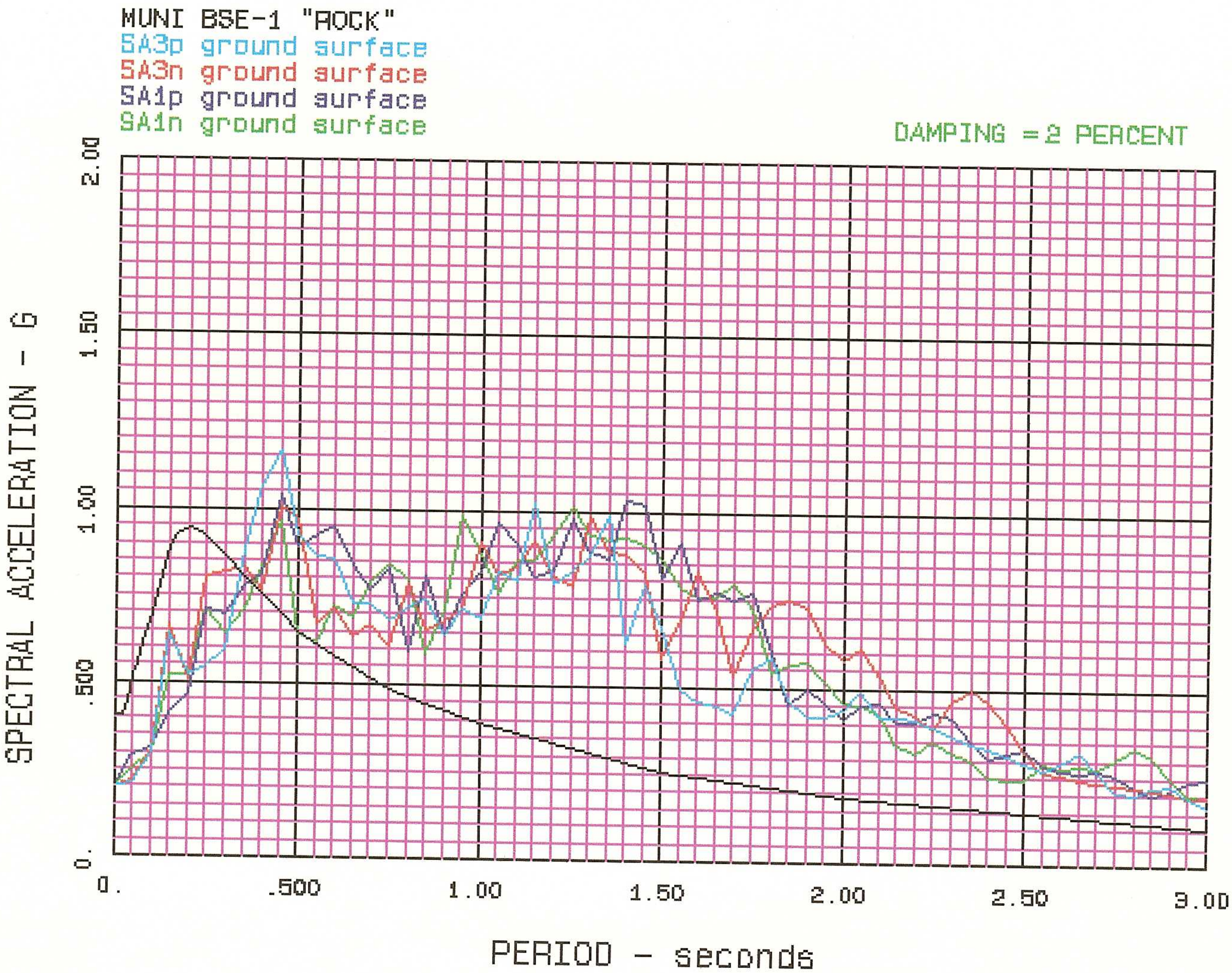

Fig.4 PUC POWER PLANT MUNI SITE
Ground Surface Spectra for BSE-1

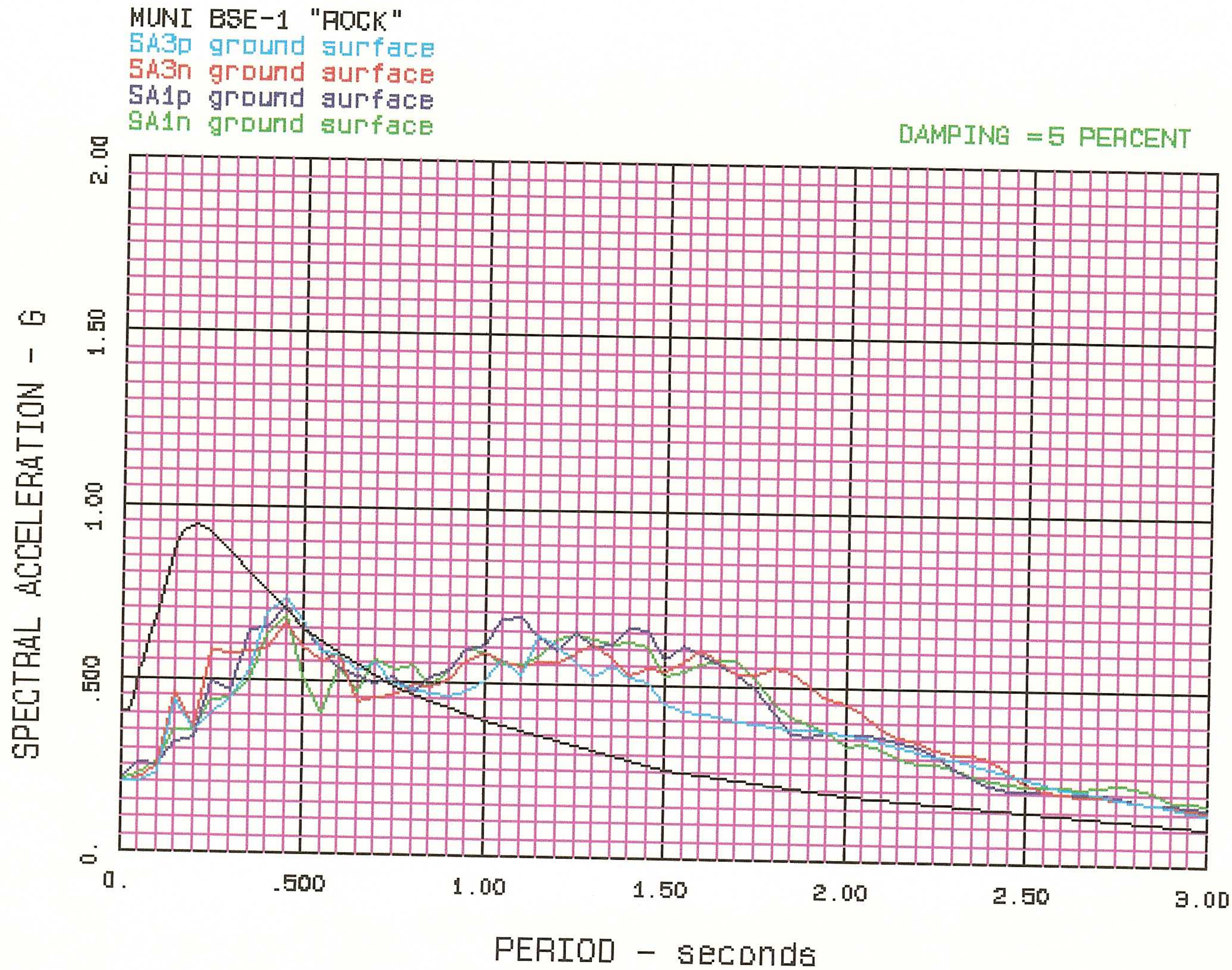

Fig.5 PUC POWER PLANT MUNI SITE
Ground Surface Spectra for BSE-1

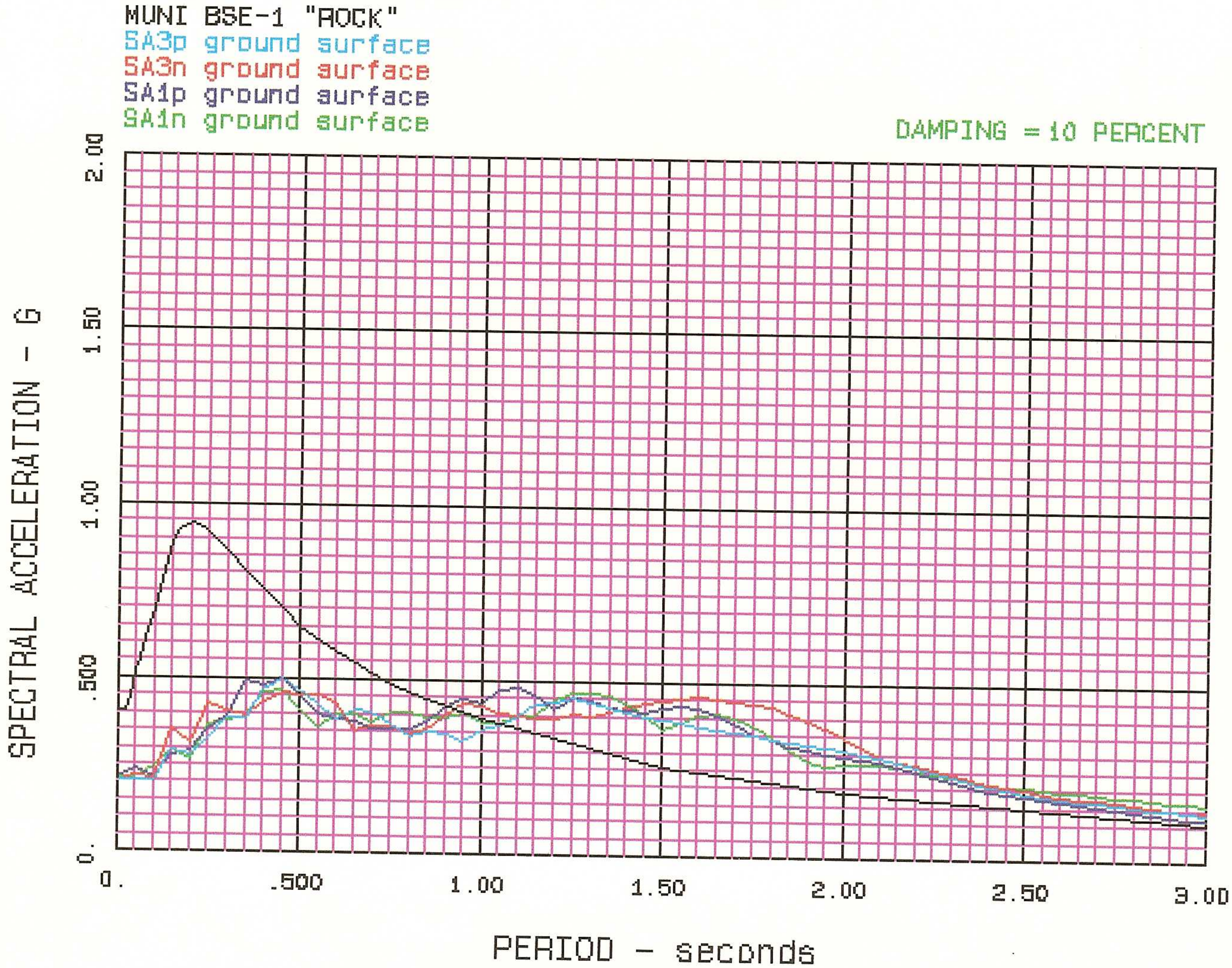

Fig.6 PUC POWER PLANT MUNI SITE
Ground Surface Spectra for BSE-1

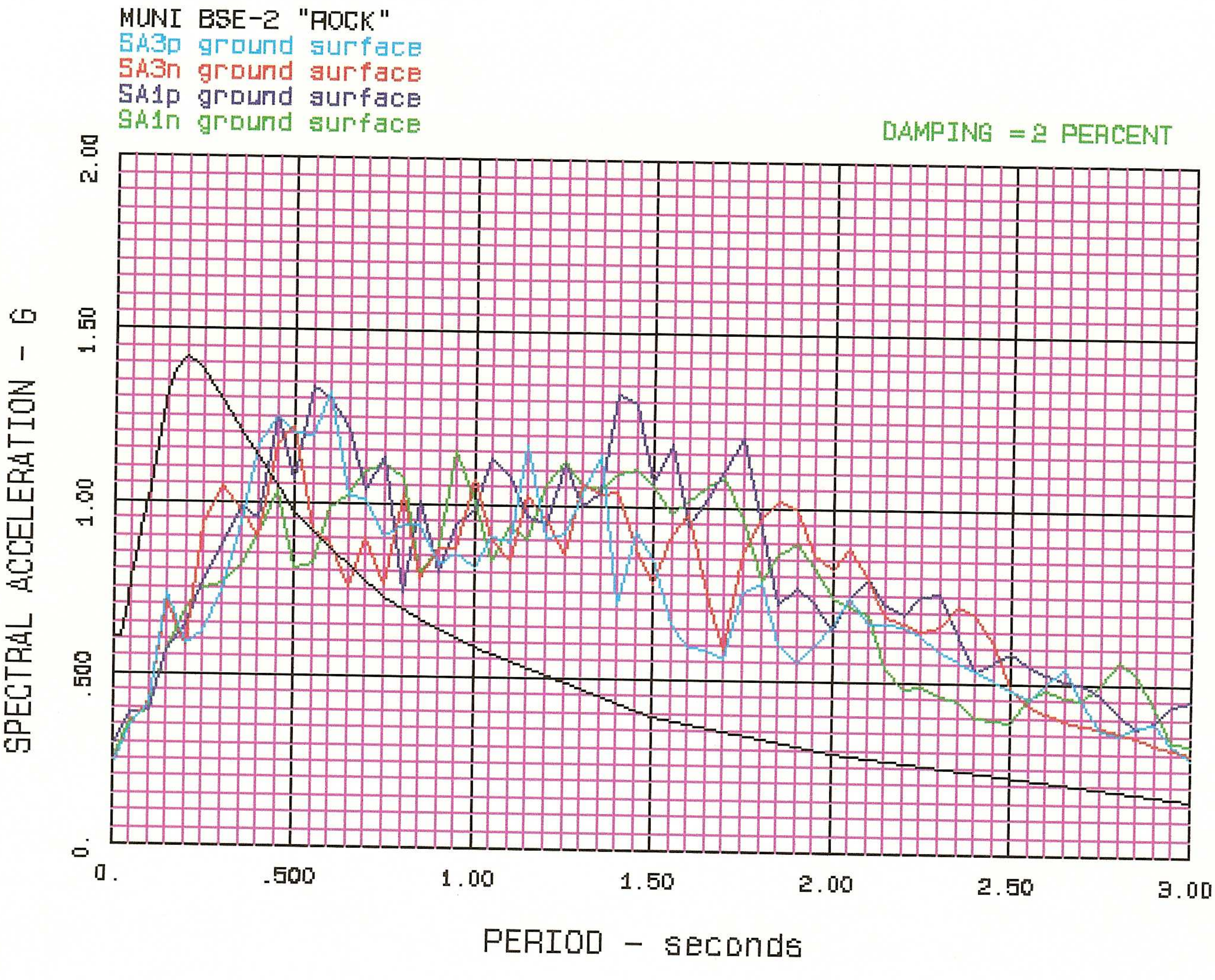

Fig.7 PUC POWER PLANT MUNI SITE
Ground Surface Spectra for BSE-2

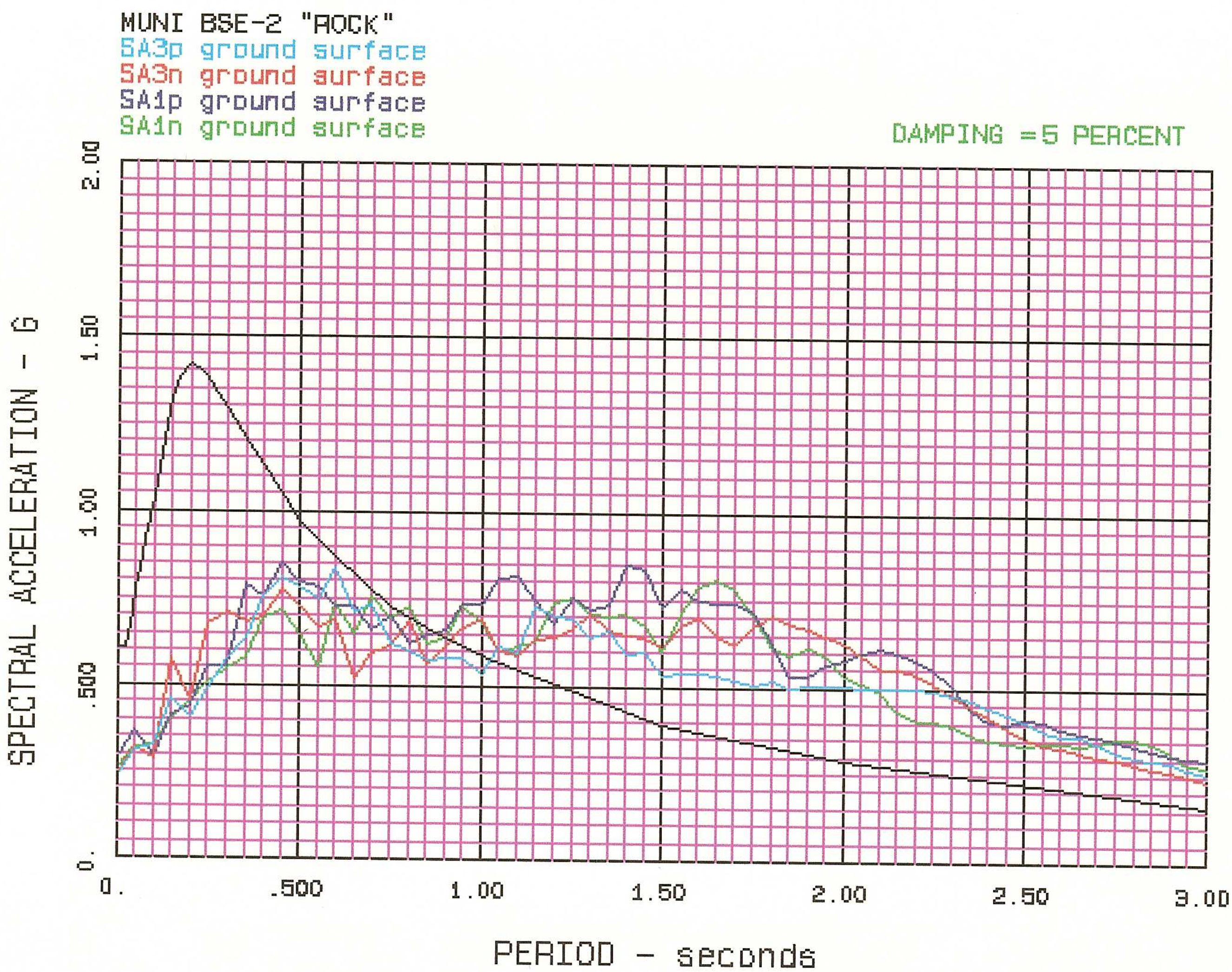

Fig.8 PUC POWER PLANT MUNI SITE
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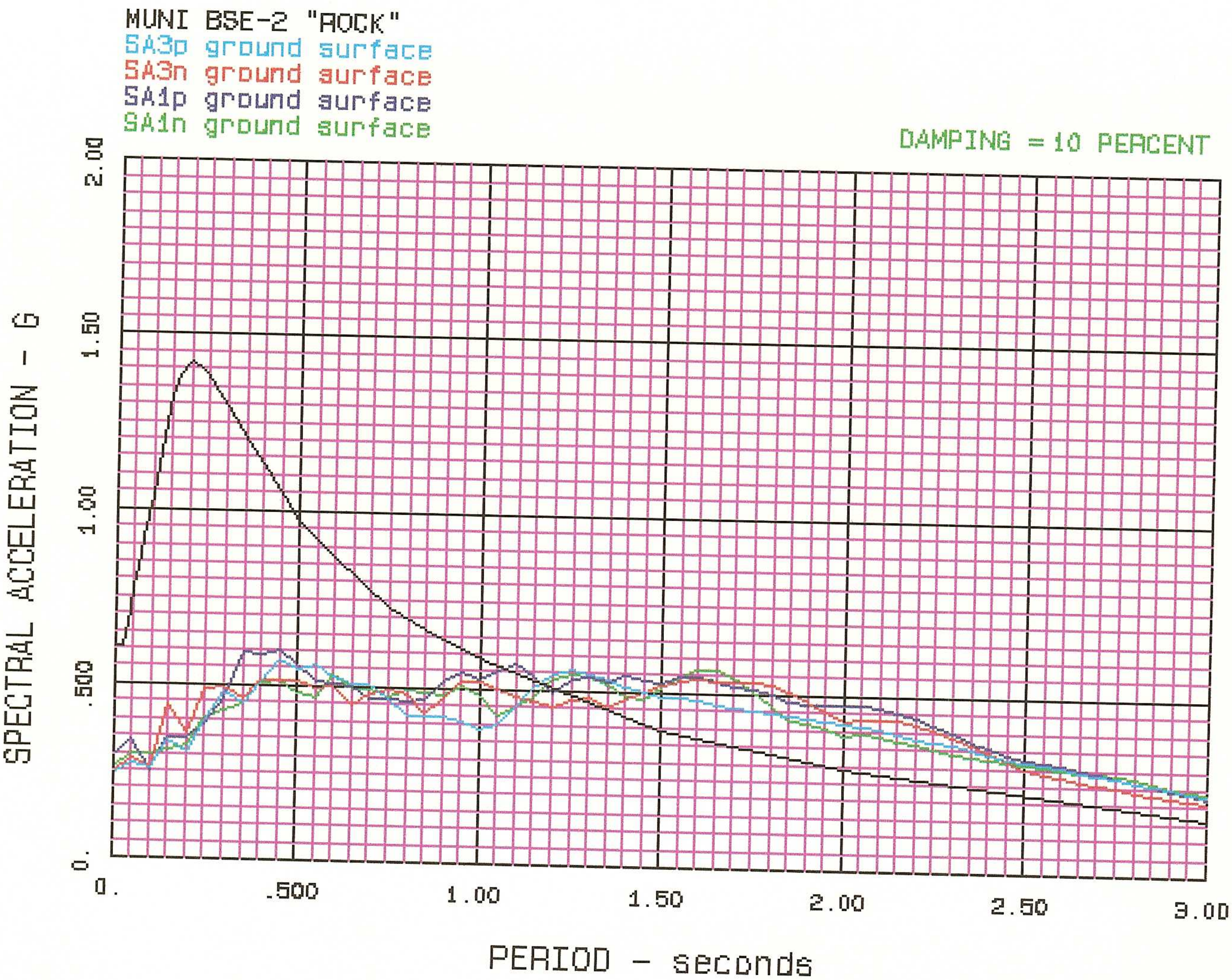

Fig.9 PUC POWER PLANT MUNI SITE
Ground Surface Spectra for BSE-2

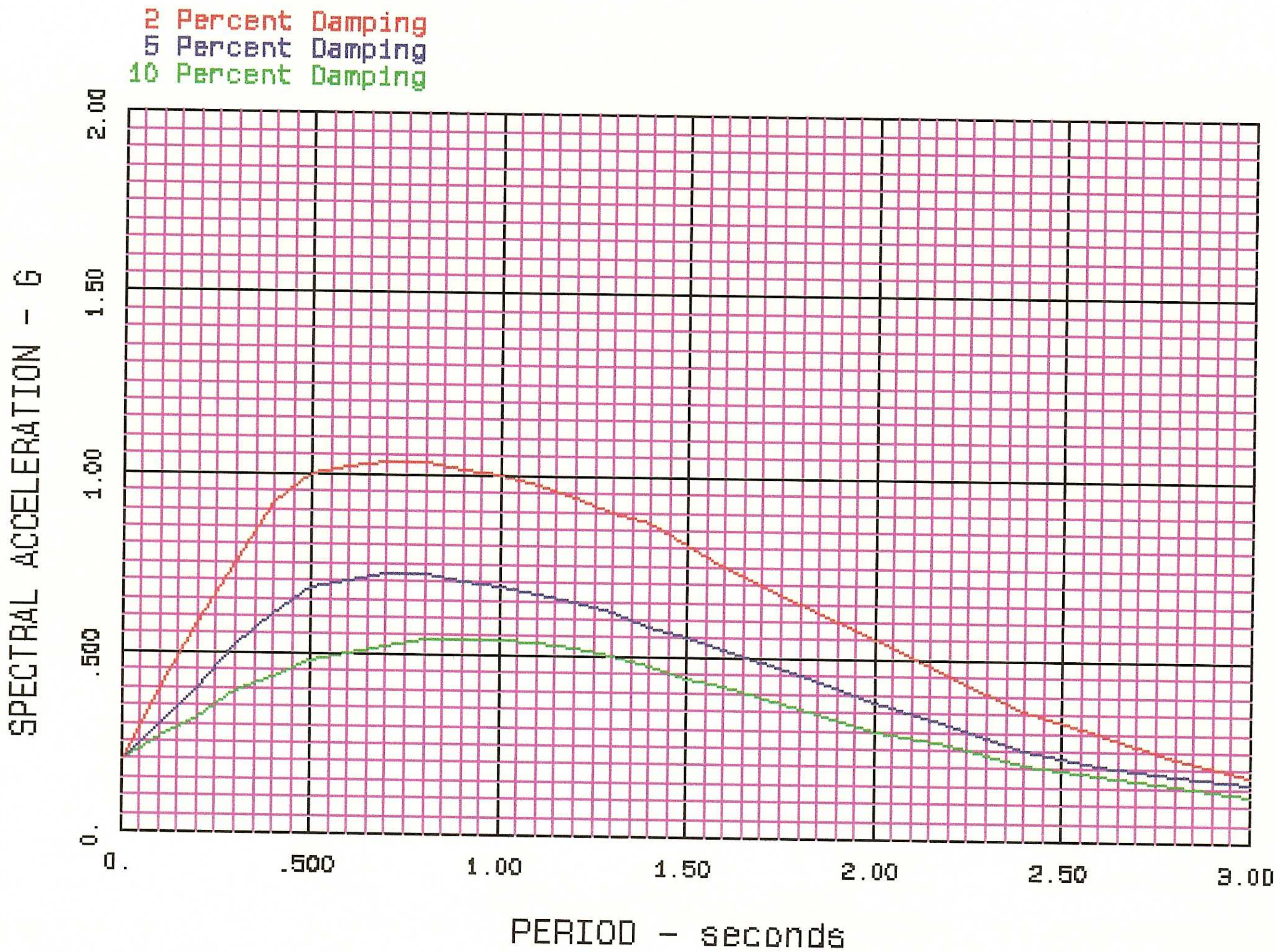

Fig.10 PUC POWER PLANT MUNI SITE
Ground Surface Spectra for BSE-1

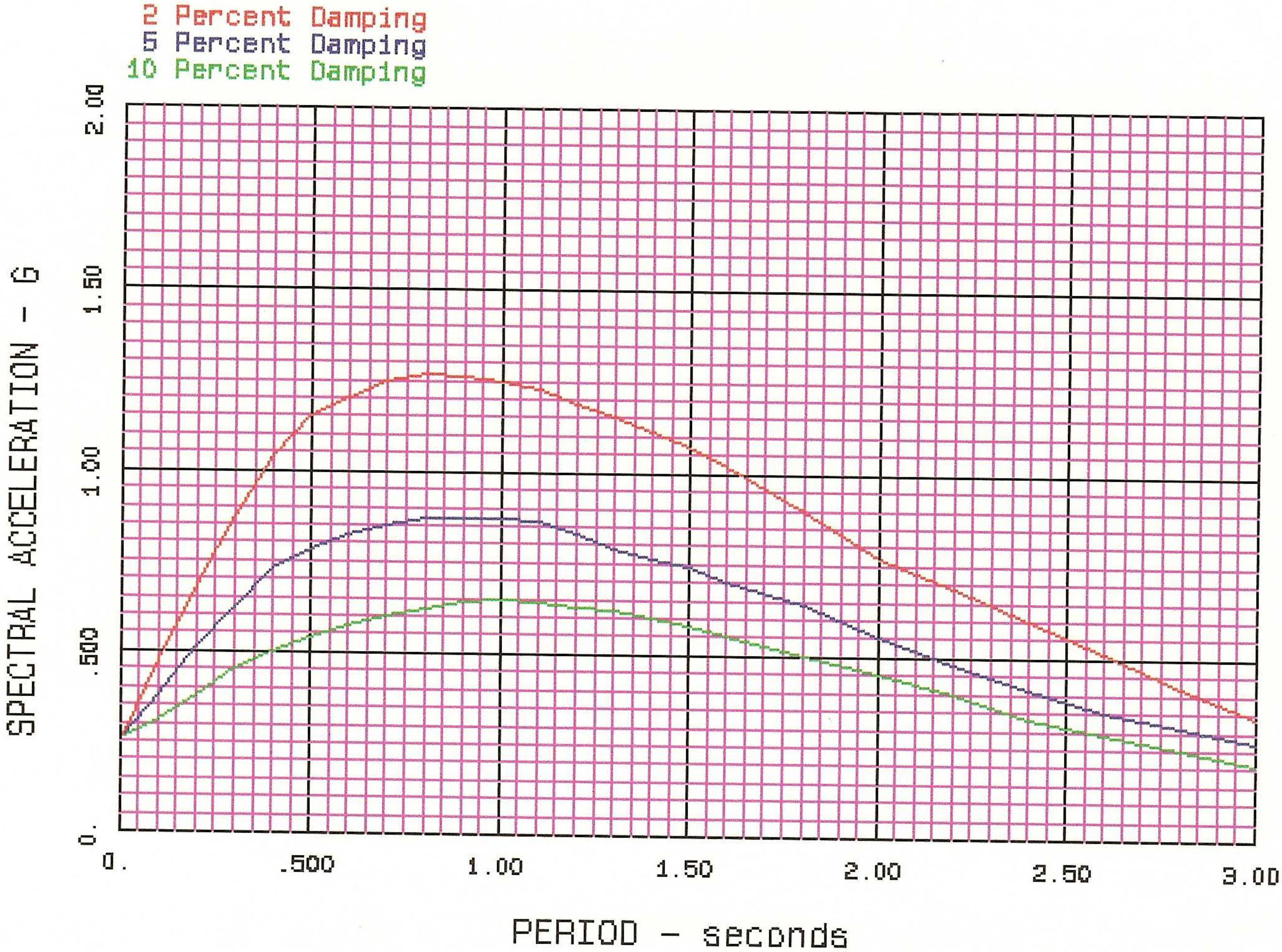

Fig.11 PUC POWER PLANT MUNI SITE
Ground Surface Spectra for BSE-2

APPENDIX C – GEOPHYSICS STUDY

GEOPHYSICAL SURVEY
MUNI SITE POWER PLANT
SAN FRANCISCO, CALIFORNIA

PREPARED FOR:
Geotechnical Consultants, Inc.
500 Sansome Street, Suite 402
San Francisco, California 94111

PREPARED BY:
Southwest Geophysics, Inc.
7438 Trade Street
San Diego, California 92121

September 16, 2005
Project No. 105062

September 16, 2005
Project No. 105062

Ms. Amy Killeen
Geotechnical Consultants, Inc.
500 Sansome Street, Suite 402
San Francisco, California 94111

Subject: Geophysical Survey
Muni Site Power Plant
San Francisco, California

Dear Ms. Killeen:

In accordance with your authorization, we have performed geophysical survey services for a proposed Power Plant to be located at the Muni Facility situated between 25th Street and Cesar Chavez Street in San Francisco, California. Specifically, our services included the performance of terrain conductivity, Sting resistivity, refraction microtremor, and downhole seismic surveys in the area of the proposed power plant. The purpose of the surveys was to provide information regarding the subsurface soil characteristics in the area of planned improvements as well as seismic design parameters for the project. This report presents the survey methodology, equipment used, analysis, and findings.

We appreciate the opportunity to be of service on this project. Should you have any questions related to this report, please contact the undersigned at your convenience.

Sincerely,
SOUTHWEST GEOPHYSICS, INC.

Patrick Lehrmann

Patrick Lehrmann, P.G., R.Gp.
Principal Geologist/Geophysicist

Hans van de Vrugt

Hans van de Vrugt, C.E.G., R.Gp.
Principal Geologist/Geophysicist

REGISTERED GEOPHYSICIST
Hans van de Vrugt
No 1012
Exp. 2/28/06
STATE OF CALIFORNIA

HV/PFL/hv
Distribution: (2) Addressee

TABLE OF CONTENTS

Page

Figures

1. INTRODUCTION

In accordance with your written authorization, we have performed geophysical survey services for the proposed power plant to be located at the Muni Facility situated between 25th Street and Cesar Chavez Street in San Francisco, California (Figure 1). Specifically, our services included the performance of terrain conductivity, Sting resistivity, refraction microtremor (ReMi), and downhole seismic surveys in the area of the proposed power plant. The purpose of the surveys was to provide information regarding the subsurface soil characteristics in the area of planned improvements, including the delineation of buried debris (i.e., concrete, metal, etc.), as well as seismic design parameters for the project. This report presents the survey methodology, equipment used, analysis, and findings.

2. SCOPE OF SERVICES

Our scope of services included:

- Review of site plans provided by your office.
- Conducting an electromagnetic terrain conductivity survey (EM31) across the project site.
- Performance of two high resolution resistivity (Sting) traverses along the southern portion of the site.
- Performance of a ReMi profile in the southern portion of the site.
- Collection of surface to downhole and crosshole seismic P-wave and S-wave velocity data at the southwest corner of the site.
- Compilation and analysis of the data collected.
- Preparation of this report presenting our findings, conclusions, and recommendations.

3. SITE AND PROJECT DESCRIPTION

The project includes the construction of a power plant on a 4-acre portion of the Muni Metro East Light Rail Vehicle Maintenance and Operation Facility. The Muni Facility is generally located east of 3rd Street, between Cesar Chavez Street and 25th Street, in San Francisco, California (Figure 1). Currently the northern portion of the site is occupied by a concrete batch plant, whereas the southern portion of the site is vacant with the exception of an empty mobile building situated at the southwest corner of the property (Figure 2). Terrain at the site is generally flat with little to no vegetation present. The perimeter of the south portion of the site is delineated by a chainlink fence. In addition, several large reinforced concrete piles were stacked at the north end of the southern study area. Figure 3 provides a general view of the project site.

4. SURVEY METHODOLOGY

The purpose of our electromagnetic and electrical resistivity surveys was to characterize the sub-surface conditions at the site. Specifically to delineate the presence of buried debris (i.e., concrete, metals, etc.). The primary purpose of the seismic surveys was to provide seismic parameters to be used in the design of the project. The following sections provide an overview of the methodologies used during our study.

4.1. EM31 Survey

A rectangular grid measuring roughly 250 feet by 470 feet was established in the southern portion of the site in order to facilitate the collection of EM31 data in this area (Figures 4a and 3b). In addition, random traverses were collected over accessible areas in the northern portion of the project site (concrete batch plant). The EM31 was synchronized with a Trimble Pro XRS Global Positioning System (GPS) for spatial control. Traverses were generally conducted along lines spaced 5 feet apart. Following the collection of the EM31 data, the data were downloaded to a laptop computer in the field and processed. The purpose of the EM31 survey was to collect terrain conductivity data across the site in order to delineate anomalous areas. The information collected was also used in siting the Sting resistivity profiles.

4.2. Sting Resistivity Survey

A high resolution resistivity survey was conducted at the site to evaluate the subsurface soil conditions as well as the presence of buried concrete and other miscellaneous debris. The resistivity data were collected along two roughly orthogonal spreads (Figure 2) with an AGI Sting resistivity meter and a Swift automatic multi-electrode system. As depicted on Figure 2, the resistivity survey was conducted on the southern portion of the site. Due to the presence of surface obstructions, including concrete pavement, the northern portion of the project site could not be surveyed.

Resistivity spreads consisting of 56 electrodes, with an electrode spacing of 5 feet was utilized; therefore, each initial spread extended 275 feet. The electrodes were driven into the ground roughly 6 inches. The area around the electrode was then moistened with salt water in order to improve connectivity. Processing of the data was accomplished through the use of a two-dimensional resistivity modeling algorithm. The plot of the measured resistivity at each set of electrodes is recorded and displayed according to the dipole-dipole resistivity model. Resistivity values are calculated for the points beneath the survey line and then integrated into a color resistivity model section. The resulting resistivity model graphically illustrates the effect of subsurface features (e.g., buried debris).

4.3. ReMi Survey

A refraction microtremor (ReMi) survey was conducted along the southern portion of the site in the area of boreholes B-10 and B-11 (Figure 2). The ReMi technique uses recorded surface waves (specifically Rayleigh waves) which are contained in background noise to develop a shear wave velocity profile of the site down to a depth, in this case, of approximately 100 feet. The ReMi data were collected using 24 vertical component geophones spaced 10 feet apart, for a total line length of 230 feet. Fifteen records, 20 seconds long, were recorded with a 24-channel Geometrics StrataView seismograph. The data were then downloaded to a laptop computer and later processed using the SeisOpt® ReMi™ software (© Optim LLC, 2005).

4.4. Crosshole and Downhole S-Wave and P-Wave Measurements

The P- and S-wave velocity measurements were obtained at two geotechnical boring locations, B-10 and B-11. Borings B-10 and B-11 were excavated to an approximate depth of 31½ feet and 101½ feet, respectively, and then cased with 3-inch diameter PVC, grouted into place with bentonite slurry. A downhole 3-component geophone with an inflatable bladder, to hold the geophone in place against the wall of the casing, was used to record P- and S-wave signals. S-waves were generated by using a downhole hammer with a spring loaded clamp. The downhole hammer was used in B-10 and the geophone was used in B-11. Crosshole shear wave data were collected at 5-foot intervals to a depth of 25 feet, and at a depth of 29 feet. A reading at 30 feet could not be performed because the casing ended at 30 feet in B-10. Additional shear wave data were collected by lowering the geophone in B-11 to depths of 35, 40, 45, 50, 60, 70, 80, 90, and 100 feet, and recording signals generated by the downhole hammer which was placed at a depth of 25 feet in B-10.

P-wave velocity data were collected by using a surface source (sledgehammer and plate) and the downhole geophone. The P-wave source was placed approximately 10 feet away from B-11. P-wave data were recorded at 5-foot intervals to a depth of 50 feet and then 10-foot intervals to 100 feet. The plate was impacted several times to produce stacked P-wave arrivals. Both S-wave and P-wave data were collected and stored using a Geometrics StrataView 24 channel Exploration Seismograph. It should be noted that the propagation path for the S-wave and P-wave are different, since the S-wave data was collected using the crosshole technique, and the P-wave data were collected using the surface to downhole method. Nevertheless, based on the results of our surveys and the information provided by your office, the seismic data collected at the site is generally representative of the site conditions.

5. RESULTS

The following is a summary of our findings:

- Electromagnetic data (EM31) reveal a relatively large area containing fairly resistive material or material of low conductivity in the southern study area. Figure 4a illustrates the extent of this area (note contour values). Areas of relatively conductivity were also encountered in the batch plant area, which is likely due to the presence of non reinforced concrete pavement and gravel fill at the surface, as well as other site interferences.

- Several areas containing conductivity highs or low resistivity materials were detected at various locations on site. Based on our field observations many of these anomalies appear to be related to surface metal debris (i.e., fence posts, fencing, wire, etc.). Figure 4b depicts the location of these features. An additional feature not the result of surface metal is the conductivity high encountered at the southeast corner of the site. Based on our field observations, this anomaly is attributed to the presence of a partially buried reinforced concrete slab.

- The results of the Sting survey, as presented in Figure 5, reveal that roughly the upper 25 feet of the site contains relatively resistive materials. In general, the more resistive material occurs on the northern portion of the south study area. In addition, pods or pockets of more resistive material within the upper 25 feet are evident in the data (note color variations in Figure 5). The results of the survey also indicate that more conductive soils are present at depth.

- The results of the ReMi survey indicate that a velocity inversion occurs in the upper 15 feet, where the average shear wave velocity decreases from roughly 1,100 feet per second (ft/s) to 800 ft/s (Figure 6).

- Figure 7 presents the velocity data and calculated Poisson's ratios derived from the cross-hole/downhole seismic survey. The results from this survey also indicate a velocity inversion over the interval from 20 to 25 feet.

6. CONCLUSIONS

As previously discussed, the purpose of our surveys was to provide information regarding the subsurface conditions at the project site and to provide seismic design parameters. Due to the presence of an active concrete batch plant at the north half of the site a very limited assessment was performed in that area. As a result, the results were generally inconclusive with regard to potentially anomalous conditions within this portion of the site. The results of the EM31 survey for the southern portion of the site revealed a relatively large area of resistive materials. Consequently, this area was further evaluated with a Sting resistivity survey.

The Sting results also indicate that relatively resistive material is present in the northern portion of the southern study area, and that the approximate depth of this material is on the order 20 to 25 feet. Based on our review of borehole logs provided by your office, this relatively resistive material generally correlates to artificial fill consisting of sandy and clayey gravels with scattered debris (i.e., concrete, wood, etc.). Also revealed in the Sting results is a near surface layer (less than 5 feet thick) and pods or pockets of more resistive material. These anomalies may represent a higher concentration of concrete or other non metallic debris in the fill.

The results of the ReMi and crosshole/downhole seismic surveys indicate that the area is underlain by two near surface soil layers. Both the ReMi and crosshole S-wave data reveal a velocity inversion where the S-wave velocities significantly decrease. The depth of the inversion occurs at 20 to 25 feet based on the crosshole data, and at 15 feet based on the ReMi data. The crosshole results are generally consistent with the Sting results and the borehole information, which reveals a geologic change at roughly 25 feet. It should be noted that ReMi results for the very near surface materials are not as precise as the crosshole results. Nevertheless, the characteristic site S-wave velocity down to a depth of 100 feet is consistent between both methods. Per IBC (International Building Code) the Vs100 for the site is 843 ft/s.

7. LIMITATIONS

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface surveying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Southwest Geophysics, Inc. should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

8. SELECTED REFERENCES

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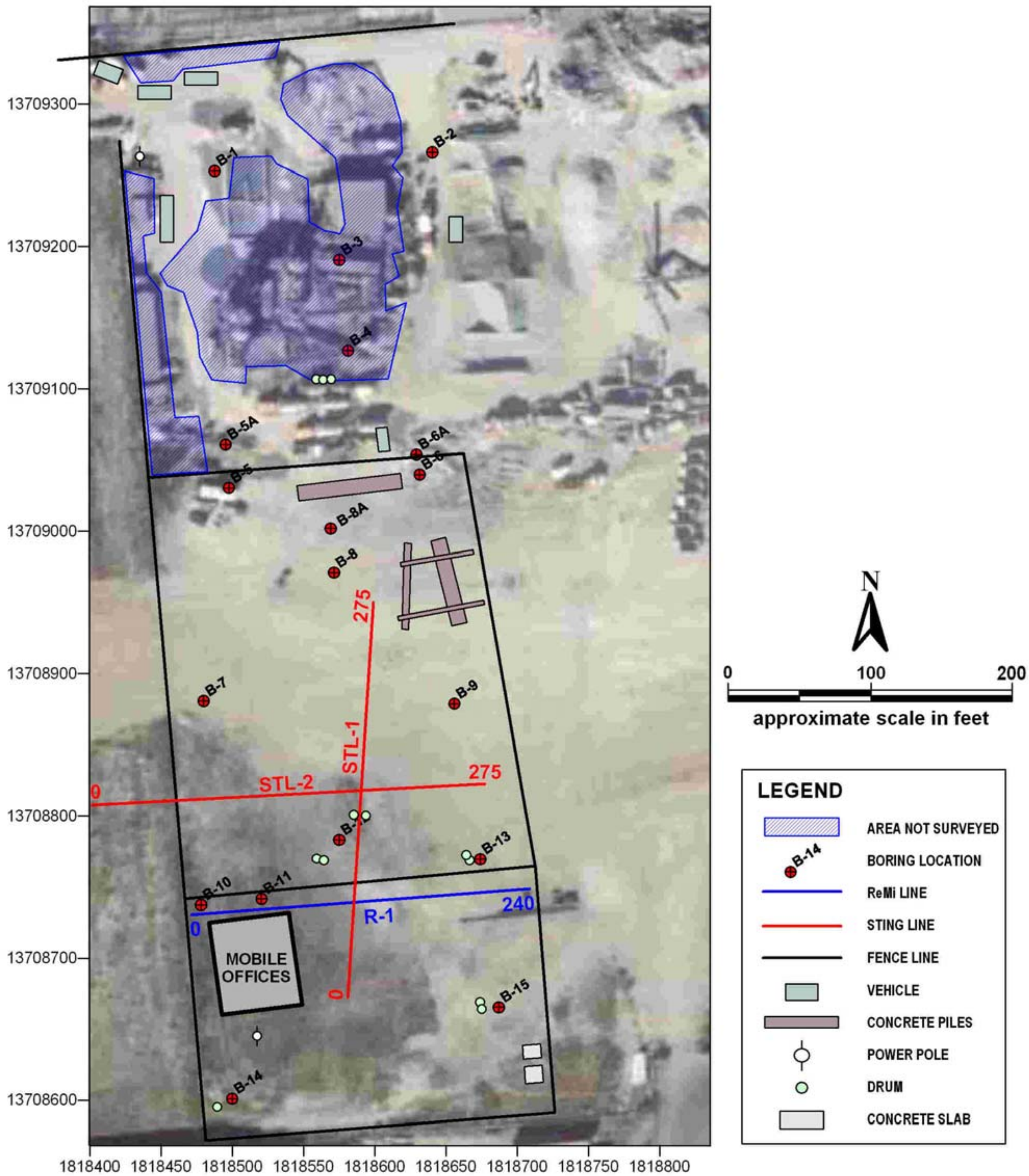
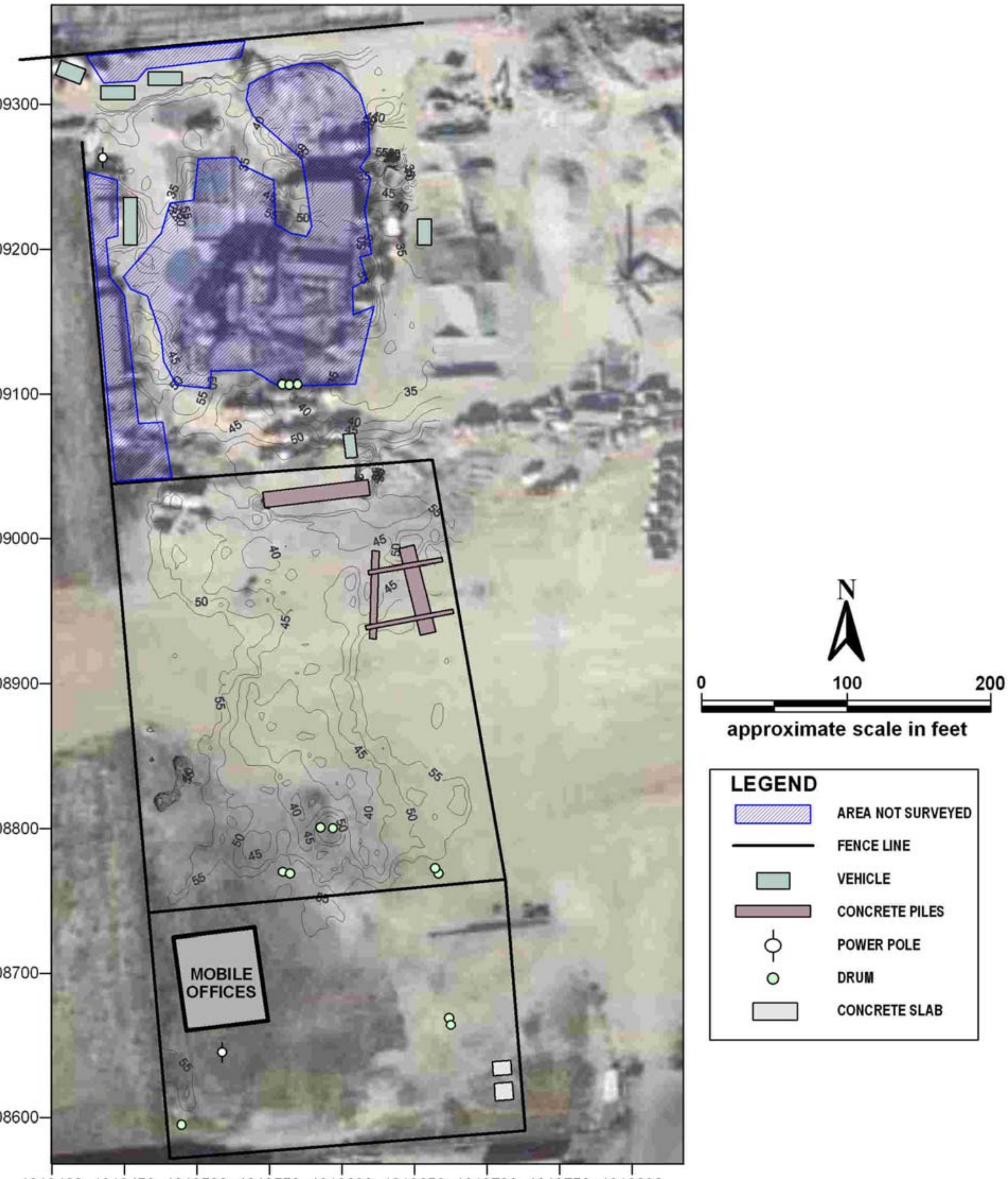
SITE LOCATION MAP	N	MUNI SITE POWER PLANT SAN FRANCISCO, CALIFORNIA	SOUTHWEST GEOPHYSICS INC.
		PROJECT NO.: 105062 — DATE: 9/05	FIGURE 1



SITE PLAN	MUNI SITE POWER PLANT SAN FRANCISCO, CALIFORNIA	SOUTHWEST GEOPHYSICS INC.
	PROJECT NO.: 105062 — DATE: 9/05	FIGURE 2

SITE PHOTOGRAPHS	MUNI SITE POWER PLANT SAN FRANCISCO, CALIFORNIA PROJECT NO.: 105062 DATE: 9/05	SOUTHWEST GEOPHYSICS INC. FIGURE 3



EM 31 CONDUCTIVITY LOWS

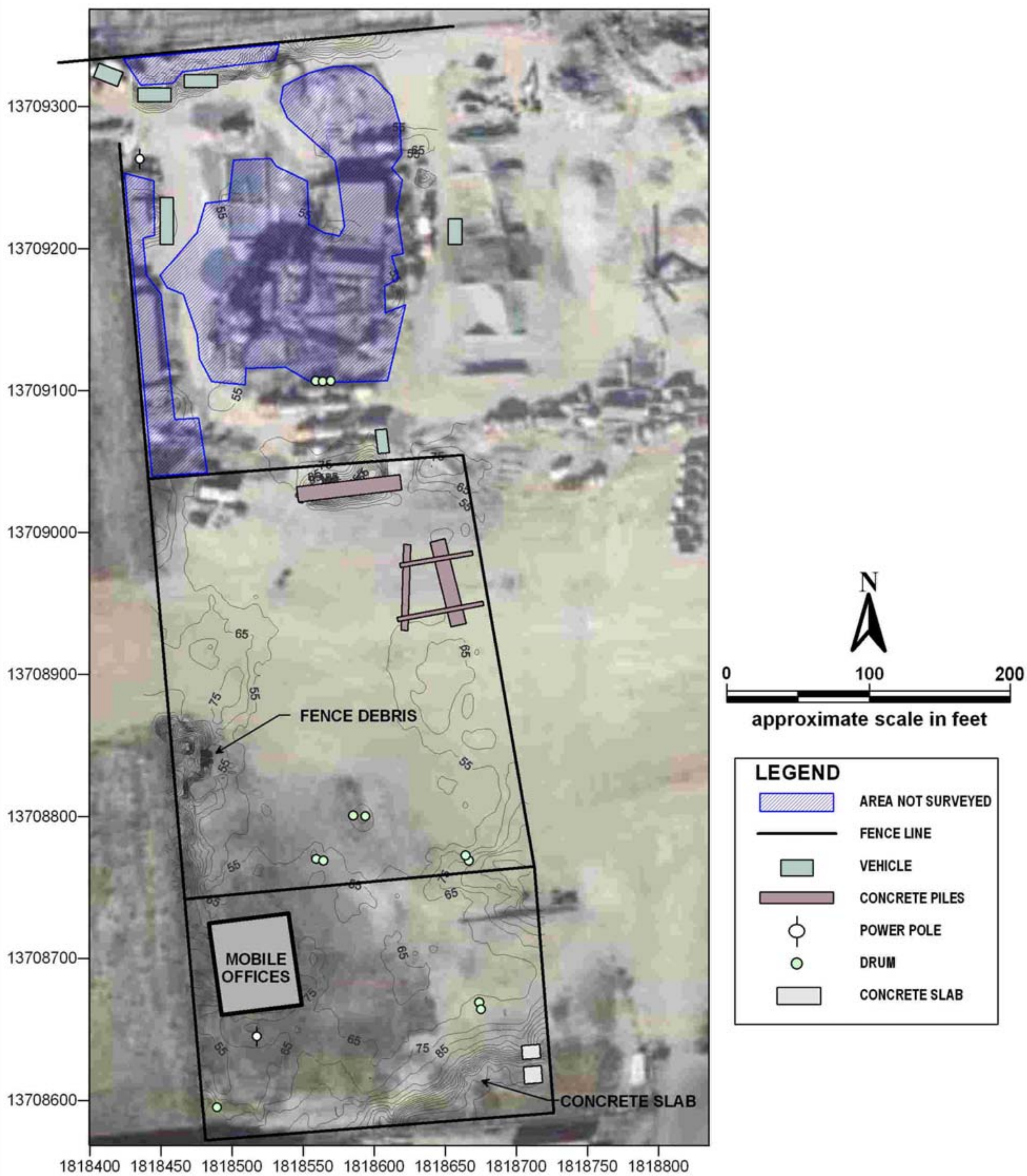
CI = 5 mMohs/m
CONTOUR RANGE 5 - 55

MUNI SITE POWER PLANT
SAN FRANCISCO, CALIFORNIA

PROJECT NO.: 105062 | DATE: 9/05

SOUTHWEST GEOPHYSICS INC.

FIGURE 4a



EM 31 CONDUCTIVITY HIGHS
CI = 10 mMohs/m
CONTOUR RANGE 55 - 280

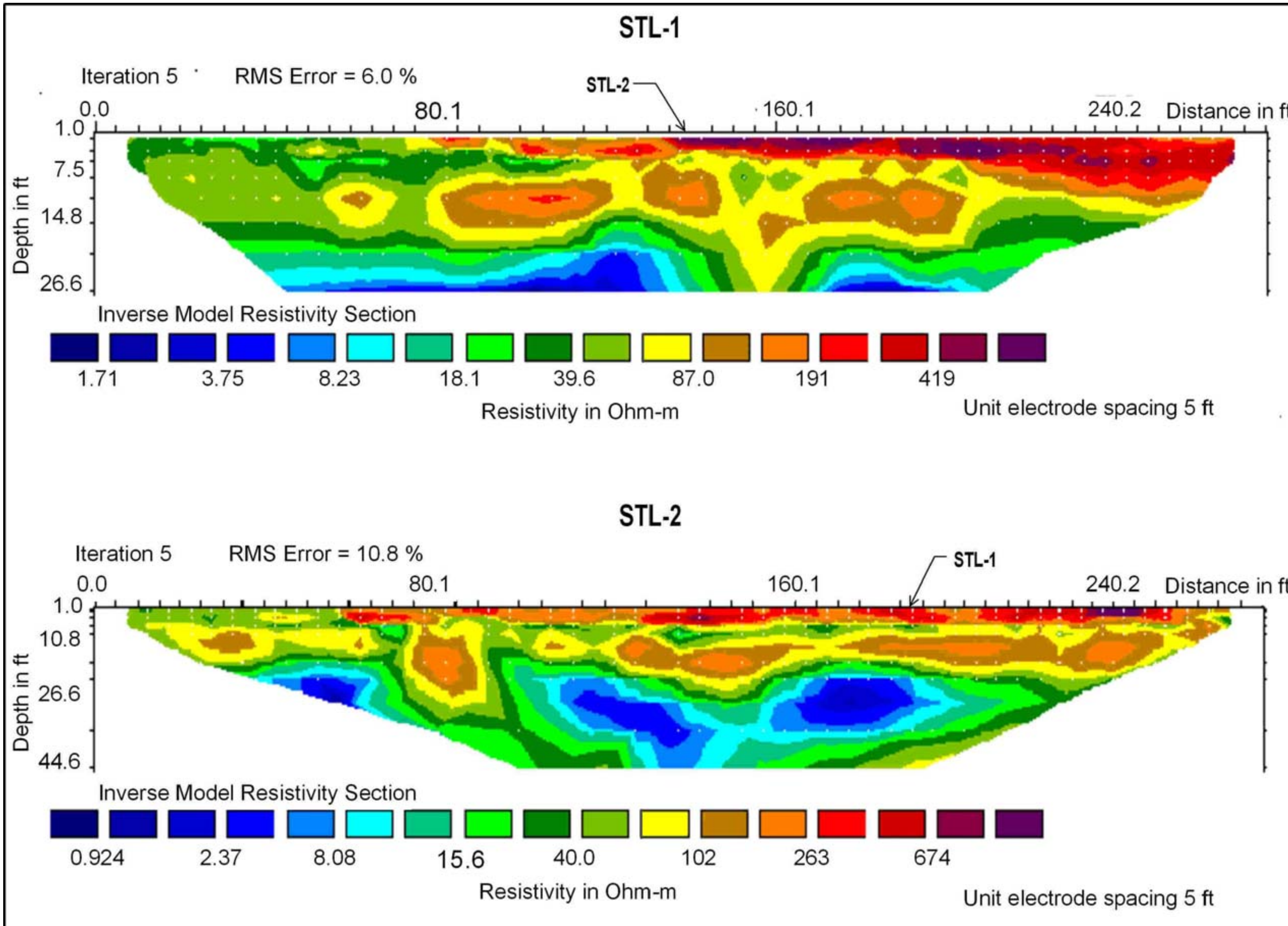
MUNI SITE POWER PLANT
SAN FRANCISCO, CALIFORNIA

PROJECT NO.: 105062

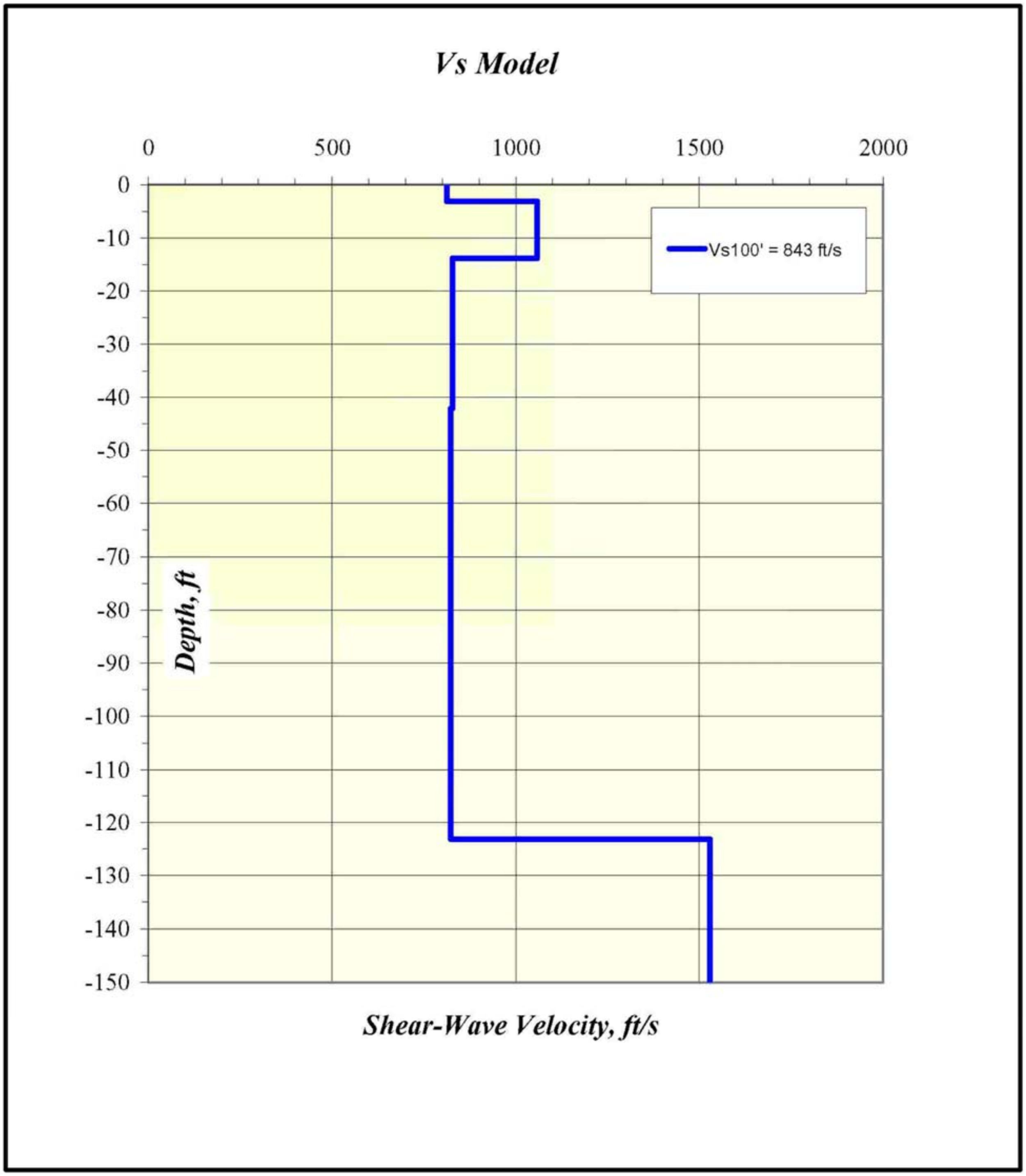
DATE: 9/05

SOUTHWEST GEOPHYSICS INC.

FIGURE 4b



STING PROFILES STL-1 AND STL-2	MUNI SITE POWER PLANT SAN FRANCISCO, CALIFORNIA PROJECT NO.: 105062 DATE: 9/05	SOUTHWEST GEOPHYSICS INC. FIGURE 5



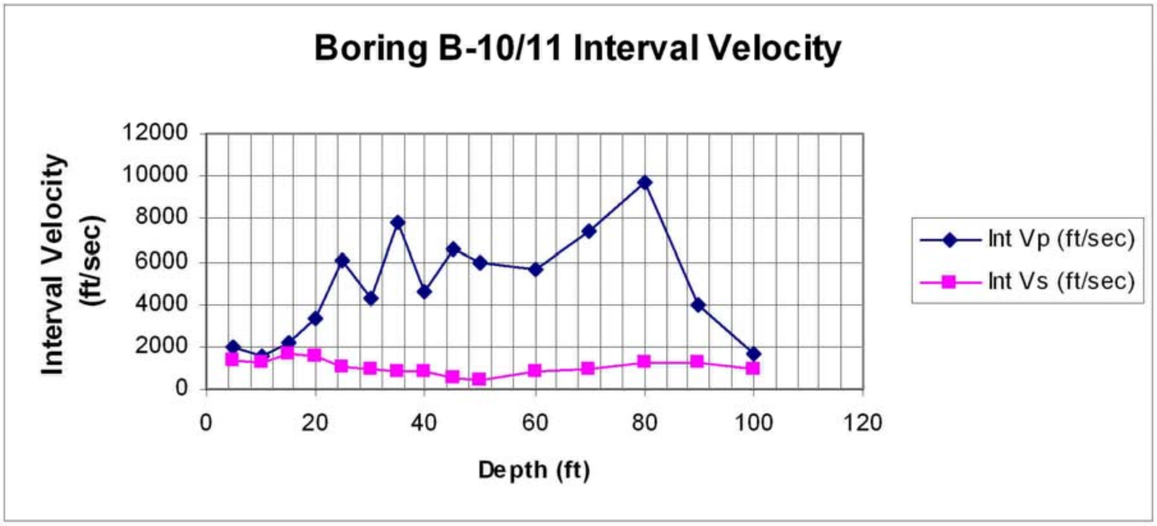
ReMi RESULTS

MUNI SITE POWER PLANT
SAN FRANCISCO, CALIFORNIA

PROJECT NO.: 105062 | DATE: 9/05

SOUTHWEST GEOPHYSICS INC.

FIGURE 6



Boring	Depth (feet)	P-wave Direct (msec)	P-wave Vertical (msec)	Ave Vp (ft/sec)	Int Vp (ft/sec)	S-wave Direct (msec)	S-wave Vertical (msec)	Ave Vs (ft/sec)	Int Vs (ft/sec)	IntVp/IntVs	Poisson's Int
B-10/11	5	5.50	2.46	2033	2033	35.00	--	1329	1329	1.53	0.13
	10	7.90	5.59	1790	1599	38.00	--	1224	1224	1.31	-0.21
	15	9.50	7.90	1898	2157	28.50	--	1632	1632	1.32	-0.17
	20	10.50	9.39	2130	3362	30.00	--	1550	1550	2.17	0.37
	25	11.00	10.21	2448	6085	45.00	--	1033	1033	5.89	0.49
	30	12.00	11.38	2635	4270	50.00	--	930	930	4.59	0.48
	35	12.50	12.02	2912	7876	56.00	11.77	849	849	9.27	0.49
	40	13.50	13.10	3054	4639	59.00	18.11	828	789	5.88	0.49
	45	14.20	13.86	3246	6537	72.00	28.45	703	484	13.51	0.50
	50	15.00	14.71	3399	5904	85.00	40.25	621	424	13.94	0.50
	60	16.70	16.47	3642	5669	87.00	52.32	669	829	6.84	0.49
	70	18.00	17.82	3928	7428	91.00	63.28	711	912	8.14	0.49
	80	19.00	18.85	4243	9669	93.00	71.02	774	1293	7.48	0.49
	90	21.50	21.37	4212	3976	97.00	78.89	824	1270	3.13	0.44
	100	27.50	27.36	3655	1668	105.00	89.24	840	966	1.73	0.25

SEISMIC CROSSHOLE/DOWNHOLE RESULTS

MUNI SITE POWER PLANT
SAN FRANCISCO, CALIFORNIA

PROJECT NO.: 105062 DATE: 9/05

SOUTHWEST GEOPHYSICS INC.

FIGURE 7